MERRIMACK RIVER BASIN
PITTSFIELD, NEW HAMPSHIRE

PITTSFIELD MILL DAM

NH 00120

NHWRB 195.11

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1978

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a 21 ft. high, 280 ft. long concrete and stone gravity dam. It is intermediate in size with a high hazard potential classification. The dam is in fair condition, is stable and has successfully withstood major floods without distress.

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.:	NH00120
NHWRB No.:	195.11
Name of Dam:	PITTSFIELD MILL
Town:	Pittsfield
County and State:	Merrimack County, NH
Stream:	Suncook River
Date of Inspection:	23 May 1978

BRIEF ASSESSMENT

The Pittsfield Mill Dam is located on the Suncook River in the center of Pittsfield and is a well-founded 21 foot high, 280 foot long concrete and stone gravity dam, including a 181 foot concrete ogee spillway. The dam is a 1967 reconstruction of a dam built in 1920, itself built on the remains of an even earlier structure. The east abutment is integrated with low level 2-bay gates operated by worm and gear handwheels, and 2-bay high level stop-log sluices. Plans of the reconstruction prepared by the New Hampshire Resources Board are available.

The drainage area is 131 square miles, and the dam normally impounds 1800 acre-feet. The dam's size classification is thus INTERMEDIATE, and because of the threat to life and property resulting from overtopping, particularly on the flanks, the hazard potential classification is HIGH.

The dam is in FAIR condition, is stable, and has successfully withstood major floods without distress. Certain findings, however, require attention. These include localized debilitated concrete in the spillway, abutments, training walls, and gate piers; open joints in granite masonry; persistent, but low volume seepage in the downstream west abutment dike; the disrepair of one low level gate; and the probable inability of the west abutment to withstand major overtopping.

The west abutment dike of the dam appears to have been overtopped by about 1.5 feet in the 1936 flood, with an estimated flow of almost 10,000 cfs. However, the Spillway Test Flood (STF) is 75,000 cfs, and since there is no real possibility of providing discharge capacity for this very large flood, other protective solutions must be investigated, in addition to development of a satisfactory warning system.

It is recommended that: the draw-down capacity of the dam be restored by gate repair; investigations should be made of solutions for preparing the west dike to withstand appreciable overtopping, and for diversion of overtopping waters into the Suncook below the downstream bridge; monitoring of seepage should be intensified; a continuing schedule for restoration of debilitated concrete and masonry should be put in hand; and warning measures, with allocation of responsibilities, should be developed and disseminated.

The above recommendations should be implemented within 1 to 2 years after receipt by the owner of the Phase I Inspection Report. Alternatives would include breaching the dam, or providing a large flood by-pass channel.

William S. Zoino, PE	James H. Reynolds, PE
NH Registration 3226	Mass. Registration 8044

PREFACE

This report is prepared under guidance contained in the "Recommended Guidelines for Safety Inspection of Dams " for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

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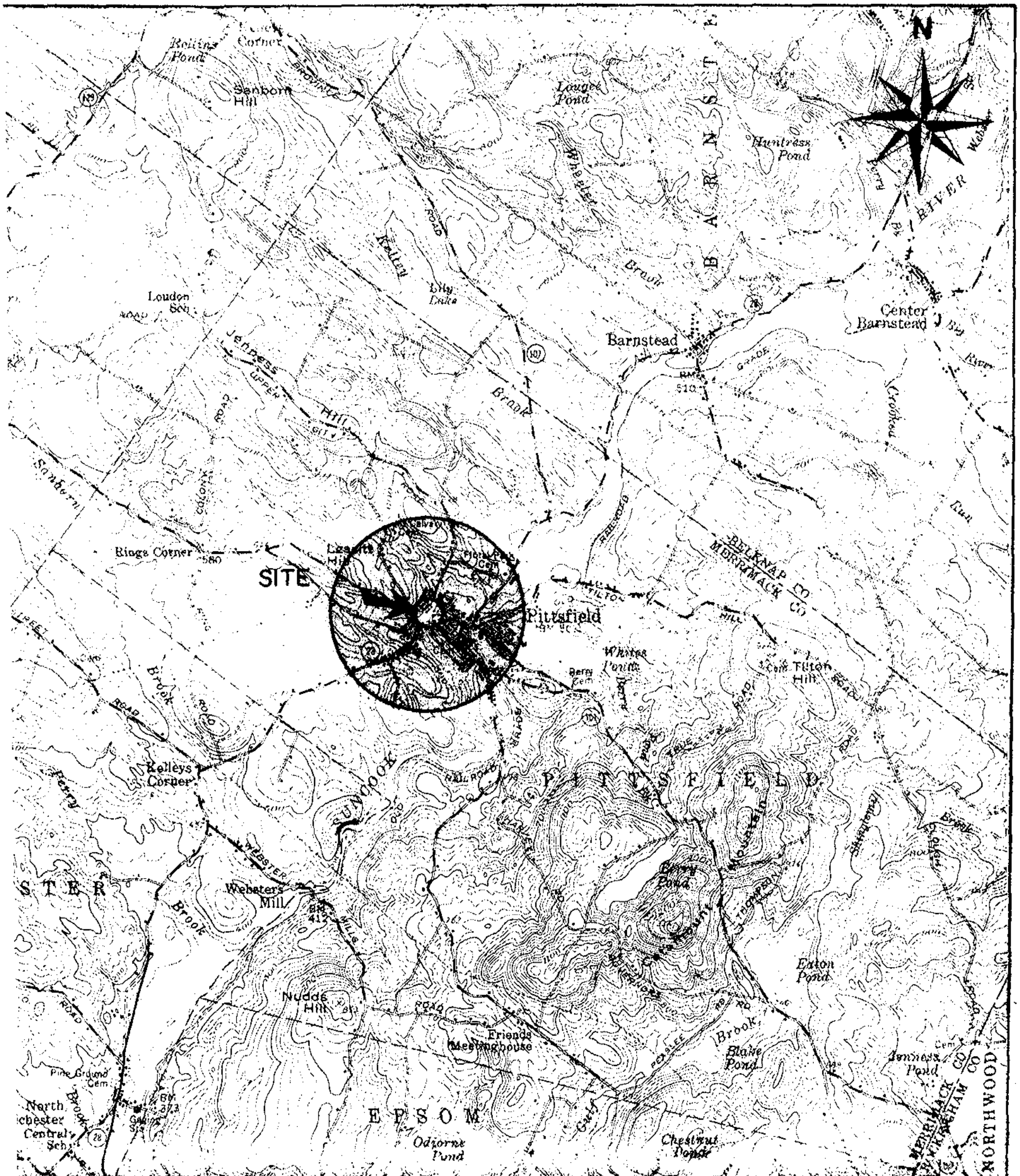
Overview from right abutment

Overview from left abutment



- SCALE -

1/2 1 1-1/2 2 miles

ROM: USGS GILMANTON, N.H. QUADRANGLE MAP

GEOTECHNICAL CONSULTANTS

NATIONAL DAM INSPECTION PROGRAM

U.S. ARMY CORPS OF ENGINEERS, NEW ENGLAND DIVISION

PITTSFIELD MILL DAM NH 00120

NHWRB 195.11

LOCUS PLAN

JULY 1978

PHASE I INSPECTION REPORT

PITTSFIELD MILL DAM, NH00120

NHWRB 195.11

SECTION 1 - PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino & Dunnicliff Associates has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to Goldberg, Zoino & Dunnicliff Associates under a letter of May 3, 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0303 has been assigned by the Corps of Engineers for this work.

(b) Purpose

(1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

(2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-Federal dams.

(3) Update, verify, and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-Federal dams in the high hazard potential category based upon location of the dams, and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dam.

1.2 Description of Project

(a) Location

The Pittsfield Mill Dam is located in the Merrimack River Basin on the Suncook River in mid-Pittsfield, at the intersection of Toure 28 and Main Street. The locus is shown on the USGS Gilmanton quadrangle, and Figure 1 presents the site in relation to the town of Pittsfield.

(b) Description of Dam and Appurtenances

The Pittsfield Mill Dam is essentially a 21-foot high concrete and stone gravity dam, which is founded, according to early documents, on rock and firm glacial till. There are references on plans to a timber mat foundation, probably the remnants of an original timber crib dam known to have existed prior to 1920. It appears from the 1961 construction plans that all new foundations were placed below the timber mat.

The right abutment of the dam, westerly of the reconstructed concrete section, is composed of large, cut granite blocks, faced by earth, transitioning to earth embankment. The east abutment is integrated with the gate structure which includes a 2-bay sluiceway section with stop-logs, and 2 deep gates operated by worm and gear hand whells. A 16-inch intake provides processing water to the Suncook Leather, Inc., a tannery located across Main Street east of the dam.

(c) Size Classification

The dam impounds 1800 acre-feet, and thus is within the INTERMEDIATE size class as defined by the "Recommended Guidelines".

(d) Hazard Potential Classification

The dam is located in downtown Pittsfield, a moderately populated and commercialized area. There is a mill, a market, and a tenament house downstream of the dam which would be damaged if the flanks of the dam were overtopped or failed. Since economic loss could be appreciable with the potential for loss of lives, the hazard potential is considered HIGH.

(e) Ownership

While the owners of the pre-1920 timber crib dam are not known, the 1920 structure was constructed for the Exeter Manufacturing Company. Subsequent records indicate a period of ownership by the Pittsfield Mills Company, and then finally the New Hampshire Water Resources Board.

(f) Operator

The operation of the dam is controlled by the New Hampshire Water Resources Board. Key officials are: Chairman George McGee, Chief Engineer, Vernon Knowlton, Assistant Chief Engineer, Donald Rapoza, and Staff Engineer, Gary Kerr.

The Board's telephone number is 1-603-271-3406. Alternatively, the Board may be reached through the State Capitol, 1-603-271-1110.

The abutting Suncook Leathers, Inc., as a user of the dam's impoundment, can make personnel available for emergency operations on coordination with the Water Resources Board. The Manager of the plant is Mr. Clifton Vulmer, and the plant's telephone number is 603-435-6678.

(g) Purpose of Dam

Originally established for hydro power generation, the dam's impoundment now serves only as a source of process water for Suncook Leathers, as a recreational resource, as a fire reserve, and to some extent at least, as a flood storage facility.

(h) Design and Construction History

As briefly noted above, prior to 1920 the site was apparently occupied by a timber crib dam serving local industry. By the fall of 1920 correspondence indicates that the foundations of the present dam were under construction. The Engineers then were H.M. Haven and William W. Crosby of Boston, the Contractor was H.P. Cummings Construction Company and the New Hampshire Public Service Commission maintained an over-view role. The dam was completed in 1921. Extensive repairs and improvements were designed by the Water Resources Board in late 1961 and are understood to have been constructed in 1962.

(i) Normal Operation Procedure

On request, after appropriate hearings if deemed necessary, the Water Resources Board will draw down the impoundment for a variety of reasons, including inspections, repairs, or desilting. On occasion, the State has drawn down some feet over the winter months to help accommodate spring runoffs.

1.3 Pertinent Data

(a) Drainage Areas - 131 sq. mi., rolling, forested and agriclutural.

(b) Discharge at Damsite - See Attached Stage - Discharge Curve, Appendix D

(1) Maximum known flood at damsite 10,800 cfs. (1936 flood) Elev. 480.05 (5.45 ft above spillway crest)

(c) Elevation (ft. above MSL)

(1) Top Dam: 478.6

(2) Maximum pool-design surcharge: Unknown

(3) Full flood control pool: Unknown

(4) Recreation pool: About 475.0

(5) Spillway crest: 474.6

(6) Streambed at centerline of dam: 457.6

(7) Maximum tailwater: Unknown

(d) Storage (acre-feet) - See Attached Storage - Elevation Curve, Appendix D

(1) Recreation pool: 1800 acre-feet (estimated as 2100-76.8X3.7)

(2) Top of dam: 2100 acre-feet

(e) Reservoir Surface

(1) Recreation Pool: 76.8 Ac (Approx.)

(2) Spillway crest: 76.8 Ac (Approx.)

(3) Flood Storage: 364 Ac

(f) Dam

(1) Type: Gravity concrete and stone on ledge and stone foundation

(2) Length: 421.7 ft (Includes 185 ft. of abutment, dike and embankment)

(3) Height: 21 ft

(4) Top Width: West Abut. - 10 ft 8 in,
East Abut. - 12 ft

(5) Side Slopes: Vary

(6) Zoning: Not Applicable

(7) Impervious Core: Not Applicable

(8) Cutoff: Unknown

(9) Grout Curtain: Unknown

(g) Spillway

(1) Type: Concrete

(2) Length of weir: 181 ft (1961 drawings show 156.7 ft)

(3) Crest elevation: 474.6 ft

(4) Gates - See item 1.3.(h) below

(5) Downstream Channel: Wide channel with exposed ledge just below dam narrowing to about 40 ft at Arch Bridge for Rte. 28 about 150 ft downstream of dam

(h) Regulating Outlets

	Gates	Weirs
(1) Invert	461.7	469.7
(2) Size	6.25 ft wide (each)	4.6 ft long (each)
(3) Description	Sluice Gates (2)	Removable stop-log weirs (2)
(4) Control Mechanism	Manual Gear Operation	Manual removal of stop-logs

SECTION 2 - ENGINEERING DATA

2.1 Design

Data on design is fragmented, and, must be inferred from correspondence, or plans for the 1961 reconstruction. Despite the paucity of data, however, key design elements are reasonably well established.

2.2 Construction

Little is known of the 1920 construction beyond the correspondence which describes Foundation Conditions.

2.3 Operation

Operational data is informal, consisting largely of ad hoc executions by the N.H. Water Resources Board in response to local residents.

2.4 Evaluation

In almost all categories, the engineering data must be considered as deficient, specifically in availability, adequacy, and validity. Nevertheless key design criteria can be recreated from surviving plans to the extent required for dam assessment.

3.1 Findings

(a) General

The dam is in fair condition with localized evidence of debilities in concrete, dike integrity, and gate operation. The dam consists of a concrete gravity ogee type spillway structure, concrete abutments, and training walls. Dual sluice gates, dual sluiceway with stop-logs and a valved outlet conduit are incorporated in the east abutment. The spillway structure is approximately 181 feet in length and 21 feet in height.

The right (west) abutment consists of concrete faced, earth filled cellular structure which slopes downward on a 2:1 slope approximately 23 feet into the impoundment area. A wing-wall approximately 24 feet in length connects this abutment to the westerly end of the spillway. A training wall is located normal to and at the end of the spillway. An "L" shaped wall with a concrete paved invert which is located downstream of the spillway is directly connected to the previously referenced training wall.

The east abutment which is located adjacent to the easterly end of the spillway is similar to the west abutment and extends approximately 38 feet into the impoundment area.

Dual sluice gates and sluiceways and a valved outlet to the mill operated by Suncook Leathers, Inc. are located within a concrete water control structure adjacent to the easterly end of the spillway and south of the east abutment.

The sluice gates are fabricated from 4 X 8 inch dimensional timbers, with maximum openings 6.3 ft X 7.2 ft. The gates are manually operated by worm gears equipped with hand wheels. the invert of these sluice gate openings are approximately 12.5 ft below the spillway crest elevation.

The sluiceway openings are 4 ft-8 in wide and 9.5 ft high. The inverts being approximately 5.8 ft below the spillway crest elevation. A 16-inch gate valve and outlet pipe is located at the extreme southerly end of this water control structure. This pipe outlets into the penstock connected to the Suncook Leather Mill.

The sluice gates were opened a small amount to determine their working condition. It was found that both wheels operating the gears were very difficult to turn, requiring much effort by several people. It appeared that this was due to some binding of debris in the sluice tracks and some misalignment, particularly in the westerly gate. Lubrication is also needed. Stop-logs are sometimes removed with hydraulic equipment in periods of emergency.

An additional feature of the dam is a small penstock from the east abutment to an old mill located about 200 ft downstream on the east bank. A tannery presently operating in the mill presently withdraws a small amount of water during the day for process water. This is controlled via a 16 in. gate valve at the dam and a pump and 8 in. line in the plant.

(b) Geology

Rock is reported to underlie both abutments and at least portions of the main dam structure. Direct observations of rock exposures, however, were limited to those seen along the base of the east abutment and along either side of the downstream pool north of Main Street.

Bedrock types known in the area are micaceous quartzites and schists of the Littleton Formation (1). The predominant rock type seen at the site was a relatively coarse grained mica schist which is presumed to be a member of the above mentioned formation. In general the rock is moderately hard and appeared only slightly weathered. It is fractured (jointed) and where visible, the major joints occur at high angles (50^{o}-75^{o}). The joints are spaced irregularly at between 6 inches and more than 3 feet apart. In most instances they appeared to be "tight", i.e. the spaces between the walls or surfaces of the joint vary from less than ½" to essentially closed.

Rock is exposed all along the base of the east abutment and drops to below water level beneath the downstream pool. The contact between the granite block wall making up portions of the abutment and the bedrock below was largely obscured by the presence of boulders and/or water and can not be directly observed. Similarly, seepage between the rock joints, although they appeared tight, could not be identified among the wet rock exposures. Major joints appeared to dip northwest at high angles to nearly vertical, striking northeast to north northeast toward the abutment.

Other outcrops in the immediate vicinity occur at both abutments of the Main Street bridge over the Suncook River and along the east bank of the upstream pool. Recent work for a sewer line along this side of the river show many large fresh fragments of coarse grained mica schist similar to that at the dam site. Presumably bedrock occurs relatively close to the surface all along this side of the reservoir. No outcrops were observed along the west side of the reservoir pool.

In summary, the predominant bedrock type seen at the Pittsfield Dam site is a mica schist. It is moderately hard, slightly weathered and sound. Prevalent joints in the rock occur at high angles and as exposed in the east abutment appeared to

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(1) Billings, Marland P., "Geologic Map of New Hampshire", The New Hampshire Planning & Development Commission, 1955.

strike northeast toward the abutment. Joints here and elsewhere in the immediate area appeared to occur at irregular intervals and were essentially "tight".

(c) Appurtenant Structures

(1) Spillway

It was observed that a continuous open horizontal joint approximately 4 feet below crest elevation extends from the west wingwall for approximately 60% of the length of the crest. It is estimated that this joint is 1-inch in width. Further observations have revealed localized spalling on the downstream face of the spillway.

(2) West Abutment and Walls

Minor spalling is in evidence on the upstream side of the west abutment. The downstream face of the wingwall between the west abutment and the spillway shows considerable evidence of efflourescence, checking and spalling. The pipe railing on the abutment is in good condition.

The connection between the training wall and the westerly end of the spillway is severely spalled. The joint between these structures has eroded to a depth of approximately 6 inches by 8 feet in vertical height. This opening is up to 1-inch in width. The low training wall located in line with the end of the spillway is severly scaled and reinforcing steel is exposed and badly stained.

Continuous seepage flows over this wall from a location 10 feet south of the dike, and 5 feet up slope from the dike. Deep holes are present, possibly from rodent infestation, although this zone has historically evidenced continuing seepage. Further surface draining from the road is also directed to the end of the training wall, aggravating the staining and concrete erosion.

(3) East Abutment and Water Control Structure

In general the east abutment is in good condition with evidence of minor hairline cracking. However the downstream interface between this abutment and the spillway has eroded to a depth of approximately 6-inches by 12 feet in vertical height. This opening is up to 1-inch in width. The concrete water control structure shows evidence of considerable localized erosion, spalling, checking, efflourescence, stalagtites and can be summarized as follows.

- The fascia above the sluice gage openings has, spalled with considerable random cracking, efflourescence and stalagtites.

- The outlet side of the intermediate support wall between the sluice gates shows evidence of severe localized concrete erosion. Concrete has completely eroded at the base of this wall for a depth of 15 inches and a height of 18-inches. Considerable cracking and efflourescence is in evidence adjacent to this location. This wall shows evidence of prior repairs.

- The end of the most westerly side wall of the sluice gate openings has a spalled area approximately 12 square feet.

- The double sluiceway opening is in good condition.

- The timber sluice gates and operating mechanisms are in good condition however the operation of the sluice gates are difficult. The joint efforts of the representative of NHWRB and three members of the inspection team opened the most easterly gate (6-inches) in five minutes. The closing procedure required the same effort. After proceeding with the operation of the west gate and after unseating, the gate became inoperable due to shaft misalignment. The misalignment was caused by stripped bolts fastening one of the bearing assemblies. Inspection revealed that this shaft misalignment has been subjected to prior maintenance.

- The cemented stone masonry wall located on the outlet side of the water control structure is in reasonably good condition with the exception of mortared joints. It is estimated that 75% of the joints in that portion of the wall between the spillway to the underside of the sluiceways are devoid of mortar. Approximately 50% of the joints in the remaining portion of the wall are devoid of mortar.

- Superficial fine surface random cracks are in evidence on the concrete deck surface of the water control structure. Hammer tapping of this concrete surface revealed localized subisdence of supporting earth.

- Portions of the top hand rail (approximately 10 feet) including a post section are missing, otherwise the railing is in good condition.

(d) Reservoir Area

The ponded area behind the dam is a typical run of the river pond, consisting of a relatively narrow lake that stretches for a considerable distance (about 4 miles) back up the river. The banks on either side slope up fairly steeply from the river resulting in a limited storage capacity relationship for increases in pond level. The pond behind the dam is estimated from the old stream bed to be about 20 feet deep and probably continues at a significant depth for some distance upstream.

Left and right shorelines were inspected to a distance of about 1,000 feet upstream of the dam, where a bridge spans the river. At a distance of approximately 300 feet upstream of the dam on the left abutment, loose fill is evident, apparently resulting from sewer line construction. A series of settlement cracks essentially parallel to the shoreline were noted. However, both the moderate height and size of the mass indicated no serious problems with respect to the reservoir. The cracks noted are believed to be settlement cracks associated with loose backfill of the sewer line.

The right or west shoreline appears to be very stable.

(e) Downstream Channel

The discharge channel downstream of the dam is somewhat wider than the spillway width above the Main St. bridge, but narrowing to about 40ft in passing through the bridge. Exposed ledge is evident below both abutments and in the bed of the channel, but does not represent a significant obstruction to the flow. After passing through the Route 28 bridge which is a granite arch structure about 40ft wide and 20ft high, the river remains channelized for several hundred feet downstream. Thereafter it resumes its natural course. Its channel has relatively steep slopes with some boulders but no significant obstacles to flow.

3.2 Evaluation

While the visual inspection revealed reasonably satisfactory information relative to an assessment of dam performance in a critical situation, it would be most desirable to inspect the ogee section in the dry, with no flow over the spillway. The suspect cavitations of the crest, and aerating sectors elsewhere in the face, suggest an examination under optimum circumstances.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

The State Water Resources Board operates the dam on an ad hoc basis, as needed, and after consultation or public hearing with the community.

4.2 Maintenance of the Dam

No specific program of maintenace is currently established, again being provided on an as-needed basis.

4.3 Maintenance of Operating Facilities

Sluices and gates, and their operating mechanisms are reasonably well maintained, except, as previously noted, for the westerly gate which evidently has chronic misalignment problems.

4.4 Warning Systems

No formal warning systemsor programs appear to exist, although coordination between the State's Water Resource Board, and the on-site Suncook Leather Co. is good. Prompt response to any emergency may thus be reasonably expected, but a formal program should be evolved, with responsibilities and sequences well defined.

4.5 Evaluation

Maintenance, operational procedures, and warning systems all require improvement and systemization.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

(a) Design Data

The existing data sources available for the Pittsfield Mill Dam include several prior inventories and inspection reports plus a complete set of design drawings prepared in 1961 by the New Hampshire Water Resources Board (NHWRB) for major repairs of the dam.

The basic characteristics of the dam are listed on the "Inventory of Dams and Water Power Developments" by the NHWRB dated 1934 and the "Data on Dams in New Hampshire" of the NHWRB dated April 1939. The existing data file also contained various correspondence from 1920-1921 during the construction of the dam. The original construction correspondence does not contain any reference to a spillway design flood. A NHWRB computation sheet from January 1957 evaluated the spillway capacity as 3890 cfs without any flow from the sluice gates and recommend that the spillway be upgraded to handle the flood of record which was 10,800 cfs. This would have required cutting down the spillway by 3.8ft and installing 3ft of flashboards. This modification was not carried out.

In 1961 the sluice gates were leaking and there was leakage through the west abutment of the dam. An emergency repair program was undertaken and the design drawings from that construction (8 sheets) provide the best and most current information available at this time. No modification to the spillway size was included in that work and thus no spillway design flood was computed.

(b) Experience Data

There is a U.S.G.S. stream flow recording gauge located downstream of the dam at North Chichester (Gauge No. 01089500). The drainage area at the gauge is 157 sq. mi. versus 131 sq. mi. at the Pittsfield Mill Dam. The gauge has records since 1918 with three years missing, or 57 years of record. The peak flow occurred in March of 1936, and was 12,900 cfs at the gauge A peak stage of 480.1ft was recorded just above the Pittsfield Mill Dam during this event which was estimated to correspond to a peak flow of 10,800 cfs. This was equivalent to 5.45 ft of head over the spillway or about 1.5ft over the west abutment dike. It is significant that the dam did not fail under these overtopping conditions.

(c) Visual Observations

The east abutment contains two low underflow sluice gates with worm gear control mechanisms, and two open weir sluice gates normally partially blocked with wooden stop-logs. In addition a small penstock feeds from the east abutment to an old mill located approximately 200 ft downstream on the east bank. The gates of the underflow sluice can be operated, although with considerable difficulty at present.

Downstream of the dam there is a stone arch bridge for Route 28 with a constricting width of about 40ft. East of the dam, the ground climbs away steeply from the abutment. At the west end, the ground level climbs slowly from the end of the dike across River Road and then climbs steeply.

(d) Overtopping Potential

The hydrologic conditions of interest in this Phase I investigation are those that are required to assess the adequacy of the dam in terms of its overtopping potential and its ability to safely allow an appropriately large flood to pass. This involves investigations to determine how the recommended Spillway Test Flood (STF) compares with the dam's discharge and storage capacities. None of the original hydraulic and hydrologic design records were available for use in this study.

Spillway Test Flood guidelines based on the size and hazard potential classifications of the dam are specified in the "Recommended Guidelines". As shown in Table 3 of the guidelines, a dam classified as INTERMEDIATE in size and HIGH in hazard potential, should be assigned an STF equal to the Probable Maximum Flood (PMF).

An estimate of the PMF was determined by using the chart of "Maximum Probable Flood Peak Flow Rates" obtained from the Corps of Engineers, NED. The Suncook River basin above the Pittsfield Mill Dam was considered to lie somewhere between the "rolling" and "flat" topography lines. The drainage basin contains a significant number of swamps and lakes to offset the hills in the area. Based on this consideration a runoff rate of 600 cfs/sq. mi. was selected, which for the 131 square miles basin represents a PMF of 78,600 cfs.

A Storage-Stage curve, contained in Appendix D, was developed to examine the impact of surcharge storage on peak flows.

The run of the river pond stretches back up the river a considerable distance. From the U.S.G.S. quad-sheet this distance was estimated to be roughly 5 miles with an average flood width of 600 feet. Thus a pond of 0.568 sq. mi. or 364 acres was assumed to provide surcharge storage that varied linearly with the depth above the spillway crest.

The Storage-Stage curve was used in conjunction with the Discharge-Stage information to modify STF to account for surcharge storage in accordance with the procedure suggested by the Corps of Engineers, NED, for "Estimating the Effect of Surcharge Storage on Maximum Probable Discharges." The result was a minimal reduction in STF from 78,600 cfs to 75,000 cfs.

The Discharge-Stage relationships were developed from the 1961 drawings of the dam and the assumption that the dike to the west including the low lying land at the end had an effective length of 185 ft.

5.2 Hydraulic/Hydrologic Evaluation

The Pittsfield Mill Dam on the Suncook River is a large structure and has been fairly well maintained over the years, but was obviously never designed to carry flows of the magnitude recommended as an STF under the current guidelines. It is worthy of note that the Flood Insurance Study that was completed in January of 1978 by the Federal Insurance Administration indicates that a 100-year flood of 9400 cfs would overtop the abutments of the dam by approximately 1 foot. There is only a remote possibility that the Pittsfield Mill Dam could ever be modified to pass the STF without overtopping. More realistically, efforts should be focused on determining the condition of the west dike and preparing it to withstand severe overtopping. No additional structures should be permitted to be constructed in the area below the dike and a plan should be developed to safely divert water which overtops the dike back into the stream below the Route 28 bridge to protect the dwellings on Route 28 downstream of the bridge.

5.3 Downstream Dam Failure Hazard Estimates

The flood hazards in downstream areas that would result from a failure of the dam were estimated through the use of the procedure set forth in "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs," Corps of Engineers, NED, April 1978. This procedure allows the attenuation of dam failure hydrographs to be accounted for in computing flows and flooding depths in downstream areas. These calculations take into account the hydraulic and storage characteristics of the stream reaches downstream of the dam.

For the purposes of these calculations, it was assumed that failure of the dam would occur when the granite and earthfill dike west of the spillway is overtopped. Thus failure is assumed when the pond level would be at elevation 478.6 ft or 4.0 ft above the spillway crest.

The Suncook River was divided into four reaches between the Pittsfield Mill Dam and Webster Mill, a settled area about three and on-half miles downstream. The first reach to about 1000 feet below the dam has several buildings located between the 440 ft and 460 ft contours on the USGS Topo map. The results of the calculation indicate little attenuation of the dam break hydrograph in this area and average flood depth of about 10.5 feet (above stream bed) to about elevation 450.5 ft. Several structures could be seriously affected in this region.

Several structures along River Road in Reach 2 are also located between elevations 440 ft and 460 ft. Flooding here is estimated to be about 11.5 feet above stream bottom or an average elevation of about 441.5 - a significant flood hazard. There is little flood hazard along Reach 3.

At the Webster Mill Road bridge structures appear to be about the 400 ft level or higher. Here flooding would occur to a depth of about 10 feet (elevation 400 ft) and damage potential would be low. Below Webster Mill the channel widens substantially in a marsh area that would probably attenuate the flood wave sufficiently to control major flood hazards below this point.

It should be noted that the estimated discharge from a dam failure when just overtopped is significantly less than the STF discharge. Thus, flood damages from an STF magnitude would exceed the damages from the failure scenario posited.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

There are no design data available for review of the structural stability of the dam and appurtenant structures. The extensive field investigation and findings do not indicate any extensive displacement and/or distress which would warrant the preparation of structural stability calculations based on assumed sectional properties and technical values.

Observations during the inspection period have revealed various minor deficiences which can be attributed to alternate freeze and thaw cycles, cavitation, seepage, quality of concrete and lack of proper maintenance.

Spillway

The open horizontal joint extending over a major portion of the spillway is the result of cavitation at a construction joint resulting from prior construction activities. The spalling of the vertical joints between the spillway and adjacent westerly training wall and the east abutment can be attributed to cavitation at construction joints at these locations. Surface spalls on the downstream face of the spillway can be attributed to cavitation.

West Abutment and Wingwalls

The spalling, checking, and efflorescence on the downstream face of the retaining wall adjacent to the spillway can be attributed to seepage and alternate freeze and thaw cycles. The spalling and exposure of reinforcing steel of the "L" shaped wall can be attributed to alternate freezing and thawing cycles accelerated by constant seepage flowing over a portion of this wall.

East Abutment and Water Control Surface

The concrete deterioration of the outlet side of the intermediate wall supporting the sluice gates can be attributed to cavitation. Other minor spalling, checking and efflourescence can be attributed to moisture intrusion and the effects of alternate freeze and thaw cycles. The subsidence of the soil foundation for the surface slab of the water control structure can be attributed to improper compaction.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The visual inspection of Pittsfield Mill Dam showed that the dam's condition is FAIR, and no deficiencies of major concern were revealed. However, the hydraulic/hydrological consequences of the Spillway Test Flood (STF) offer important implications.

As was shown, the dam has undergone an earlier overtopping of 1.5 feet over the west dike, but remained well founded, without significant structural distress as might be expected of the massive concrete character of the dam proper.

Nevertheless, in face of an STF 7.5 times greater than the flow of record, the safety of the most vulnerable component, the west dike, cannot be assured.

(b) Adequacy of Information

The information available is adequate as a basis on which to assess dam performance.

(c) Urgency

The dam is in no immediate danger, but gate repairs in particular should be implemented within a 1 to 2 year period after owners' receipt of the Phase I Inspection Report. Other repairs and preventive maintenance should be programmed on a continuing basis.

(d) Need for Additional Information

At this time, there is no evident need for additional information.

7.2 Recommendations

The draw down capacity of the dam is seriously deficient because of the inoperative gate. The westerly gate shaft bearing blocks should be repaired within the recommended time frame of 1 to 2 years.

Preliminary engineering studies should be initiated to investigate solutions for preparing the west dike and downstream apron to withstand appreciable overtopping under an appropriate design flood condition, and for the safe diversion of overtopping waters back into the Suncook below the downstream bridge.

7.3 Remedial Measures

(a) Alternatives

Alternatives to these recommendations are limited to breaching the dam, or constructing a major flood by-pass channel outletting into the Suncook below Pittsfield.

(b) O & M Maintenance Procedures

Intensified monitoring of the chronic west dike seepage should be initiated, not less than once per month with particular attention being given to changes in a program of continuing repairs should be developed, particularly on debilitated concrete.

Similarly, formal procedures for dam operation and warning dissemination should be developed, with clearly assigned responsibilities between the owners, local officials and public safety personnel, and the operators of the adjacent mill.

APPENDIX A

VISUAL INSPECTION CHECK LIST

INSPECTION TEAM ORGANIZATION

Date:	23 May 1978-8:45 A.M.
Project:	NH00120 Pittsfield, NH Suncook River NHWRB 195.11
Weather:	Sunny, warm

Inspection Team

James H. Reynolds	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZDA)	Team Captain
William S. Zoino	GZDA	Soils
John E. Ayres	GZDA	Geology
Nicholas A. Campagna	GZDA	Soils
Andrew Christo	Andrew Christo Engineers, Inc. (ACE)	Structural and Concrete
Paul Razgha	ACE	Structural & Mech.
Richard L. Laramie	Resource Analysis, Inc.	Hydrology

State Official Present

Gary Kerr, NH Water Resources Board

VISUAL INSPECTION CHECK LIST

PROJECT: Pittsfield Mill Dam NH00120
NHWRB - 195.11
Suncook River

DATE: May 23, 1978

Top of dam elevation	100.33 (Local = 478.36 USGS)
Spillway crest elevation	96.33 (Local)
Current Pool Elevation	96.7 (Local)
Current Impoundment	1700 acre feet

AREA EVALUATED	BY	CONDITION & REMARKS
EMBANKMENT DIKE, RESERVOIR SLOPES		
Surface cracks	[illegible]	None on dams; moderate cracking of loose backfill for new sewer line, 300 ft north of dam on east bank of reservoir
Movement or settlement of crest		None
Lateral Movement		None
Vertical Alignment		Good
Horizontal Alignment		Good
Granite Block Facing		Fair, repointing desirable
Trespassing on slopes		Not excessive
Sloughing or erosion of slopes		Localized and moderate on right upstream embankment
Riprap		None upstream; locally desirable on right upstream embankment section. Increased rip rap area desirable at downstream face of stilling basin, opposite right abutment.
Unusual movement, toe vicinity		None

VISUAL INSPECTION CHECK LIST - Cont.

AREA EVALUATED	BY	CONDITION & REMARKS
Seepage	MC AHL	a) Seepage spilling over right abutment downstream retaining wall, issuing from holes, possible rodent infestation on downstream slope 5 ft west of right spillway wall. Water not discolored. b) Evidence of seepage over top of rock in downstream channel, west of toe of east abutment training wall. Clear water. c) Surface drainage in pipe discharging water over right abutment downstream retaining wall.
Over growth		Moderate; no trees
Piping or boils		None
Foundation Drainage Feature		No weep holes in downstream retaining walls.
SPILLWAY	DP JC	Longitudinal joint 60% + length of spillway-spalled 1" wide x 1" deep, localized erosion
WEST ABUTMENT		
Upstream face		Hairline cracking, minor spalling
Pipe railing		Good condition
Wingwall - downstream face		Spalling, checking, efflourescence. No weep holes observed.

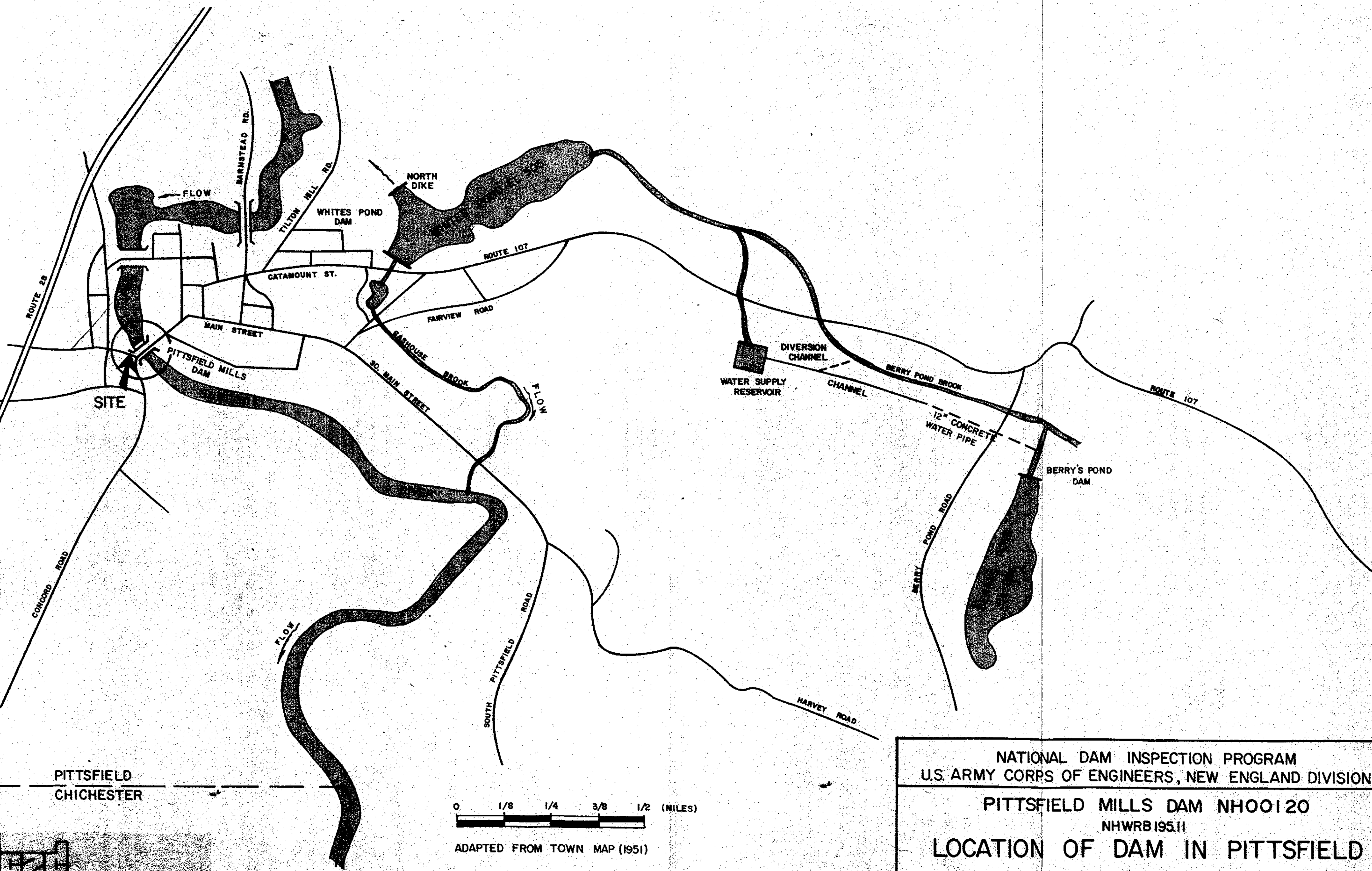
Pittsfield Mill Dam NH00120

VISUAL INSPECTION CHECK LIST - Cont.

AREA EVALUATED	BY	CONDITION & REMARKS
WEST ABUTMENT-cont.		
Training wall to spillway Vertical joint	KC	Joint spalls
Low training wall		Severely scaled, reinforcing steel exposed, discoloration, no weep holes. Seepage flowing over top of wall.
Concrete paved apron		Could not be observed due to turbulence.
EAST ABUTMENT AND WATER CONTROL STRUCTURE		
East abutment		Hairline cracking
Abutment to spillway (Vertical joint)		Joint spalls
Water control structure Fascia above sluice gate openings		Spalled, random cracking, efflourescence and stalagtites
Intermediate support wall between sluice gates		Severe localized erosion, random cracks wide to medium and efflourescence.
Inlet end of sluice gate openings		Moderate to severe surface spalling
Deck surface		Fine random cracks
Sluice gates		
Timber gates		West gate malfunctioned
Rack gears		Good condition
Worm gears & wheels		Good condition
Gear shafts		Good condition
Bearings & Housings		Bearing housing on west gate malfunctioned, gate inoperative

APPENDIX B

NOTE: Drawings B-4 through B-11 use an arbitrary datum. Elevation 100.0 = 478.3 ft MSL



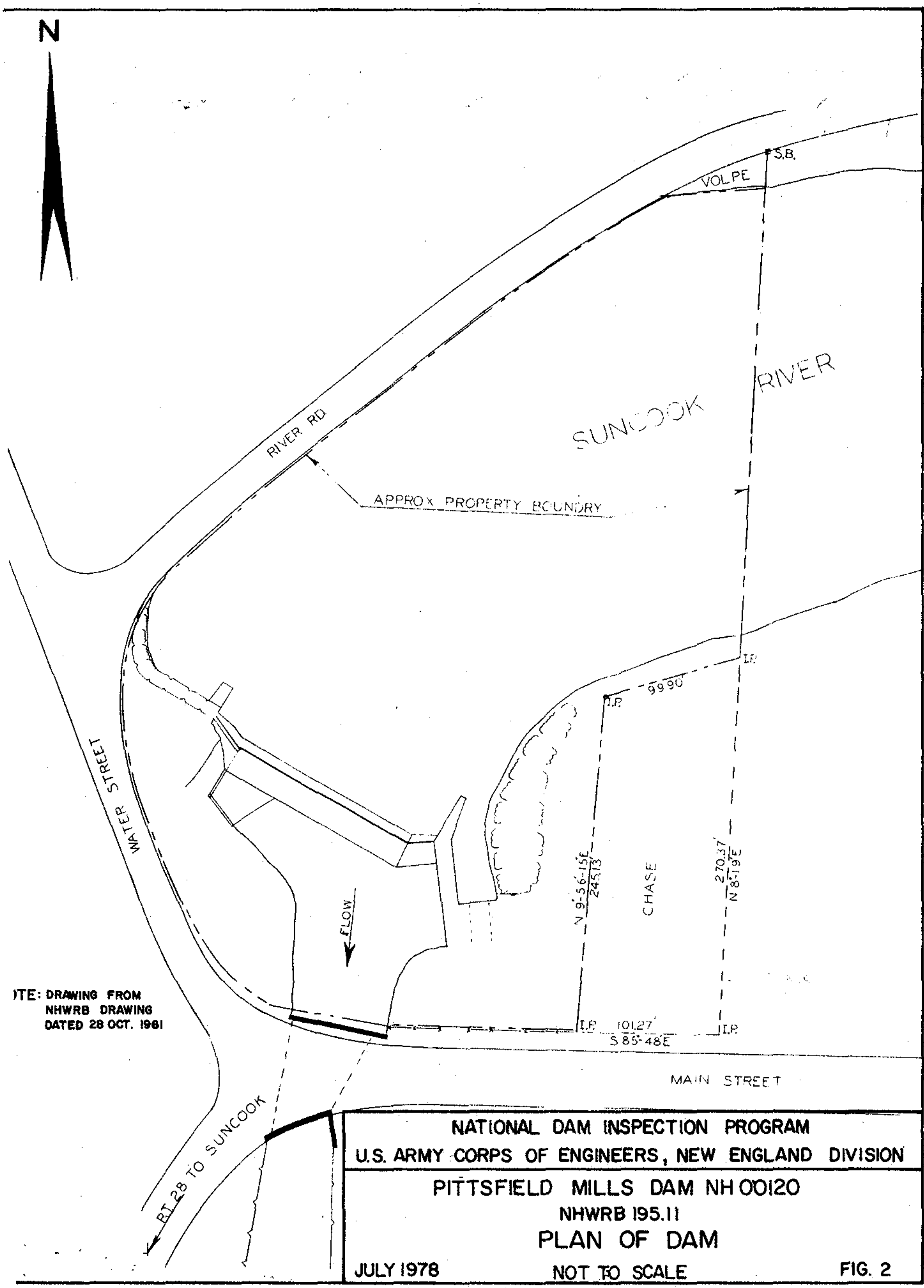
NATIONAL DAM INSPECTION PROGRAM
U.S. ARMY CORPS OF ENGINEERS, NEW ENGLAND DIVISION

PITTSFIELD MILLS DAM NH00120

NHWRB 195.11

LOCATION OF DAM IN PITTSFIELD

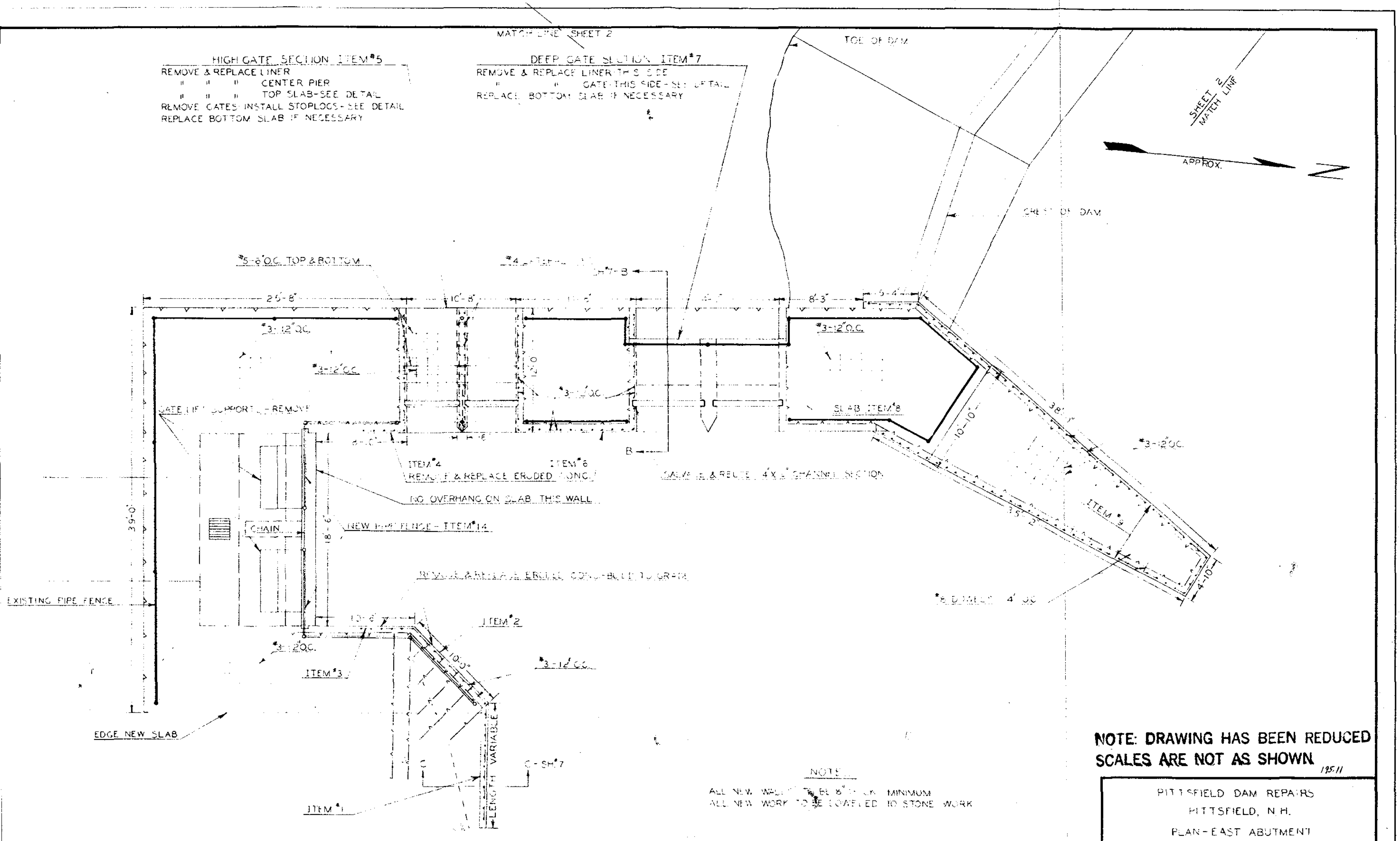
JULY 1978 | SCALE: AS NOTED | FIG. 1



NATIONAL DAM INSPECTION PROGRAM
U.S. ARMY CORPS OF ENGINEERS, NEW ENGLAND DIVISION

PITTSFIELD MILLS DAM NH 00120
NHWRB 195.11
PLAN OF DAM

JULY 1978 NOT TO SCALE FIG. 2



NOTE: DRAWING HAS BEEN REDUCED SCALES ARE NOT AS SHOWN

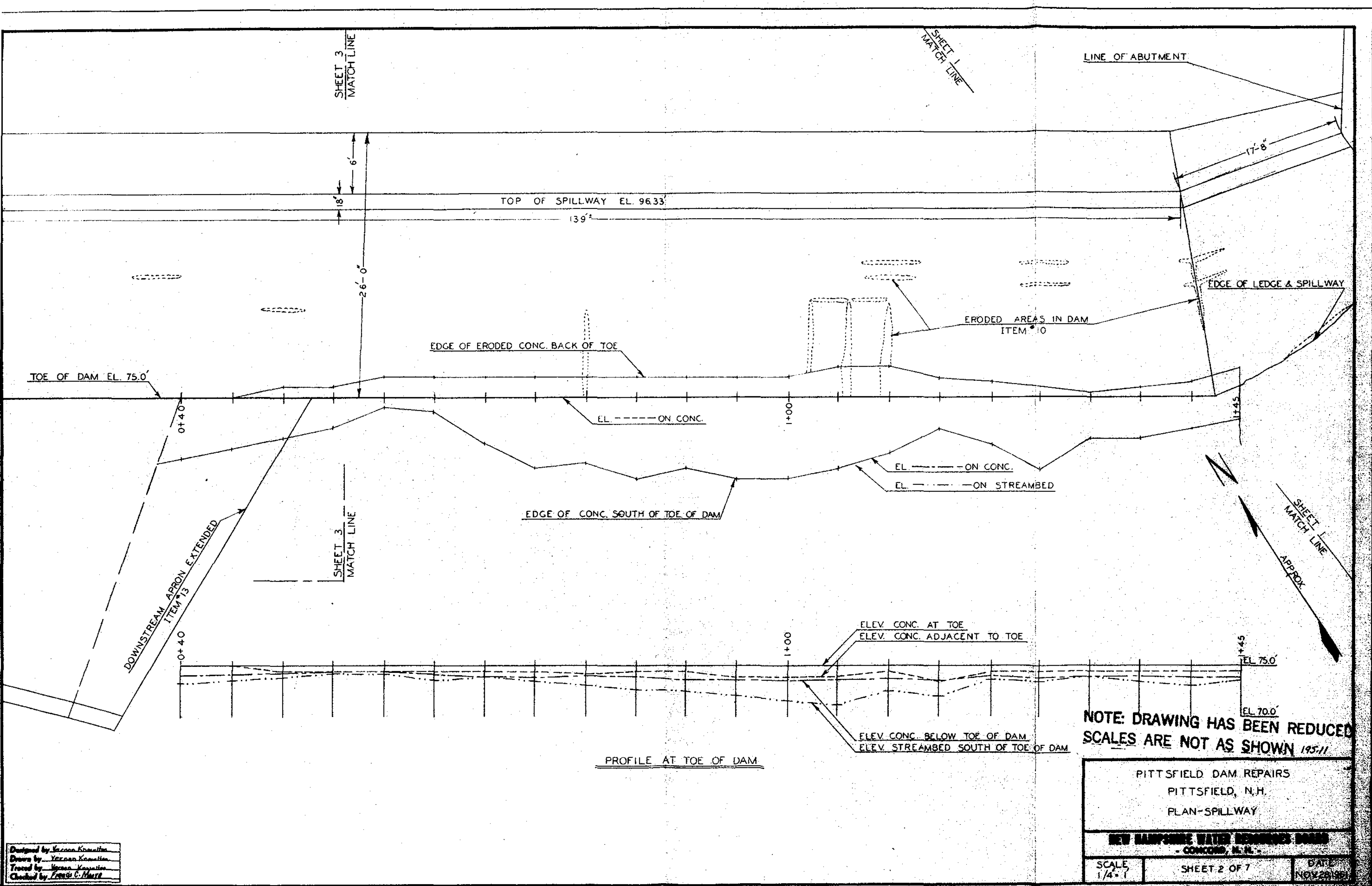
Designed by Vernon Knowlton
Drawn by Vernon Knowlton
Traced by Vernon Knowlton
Checked by Francis C. Moore

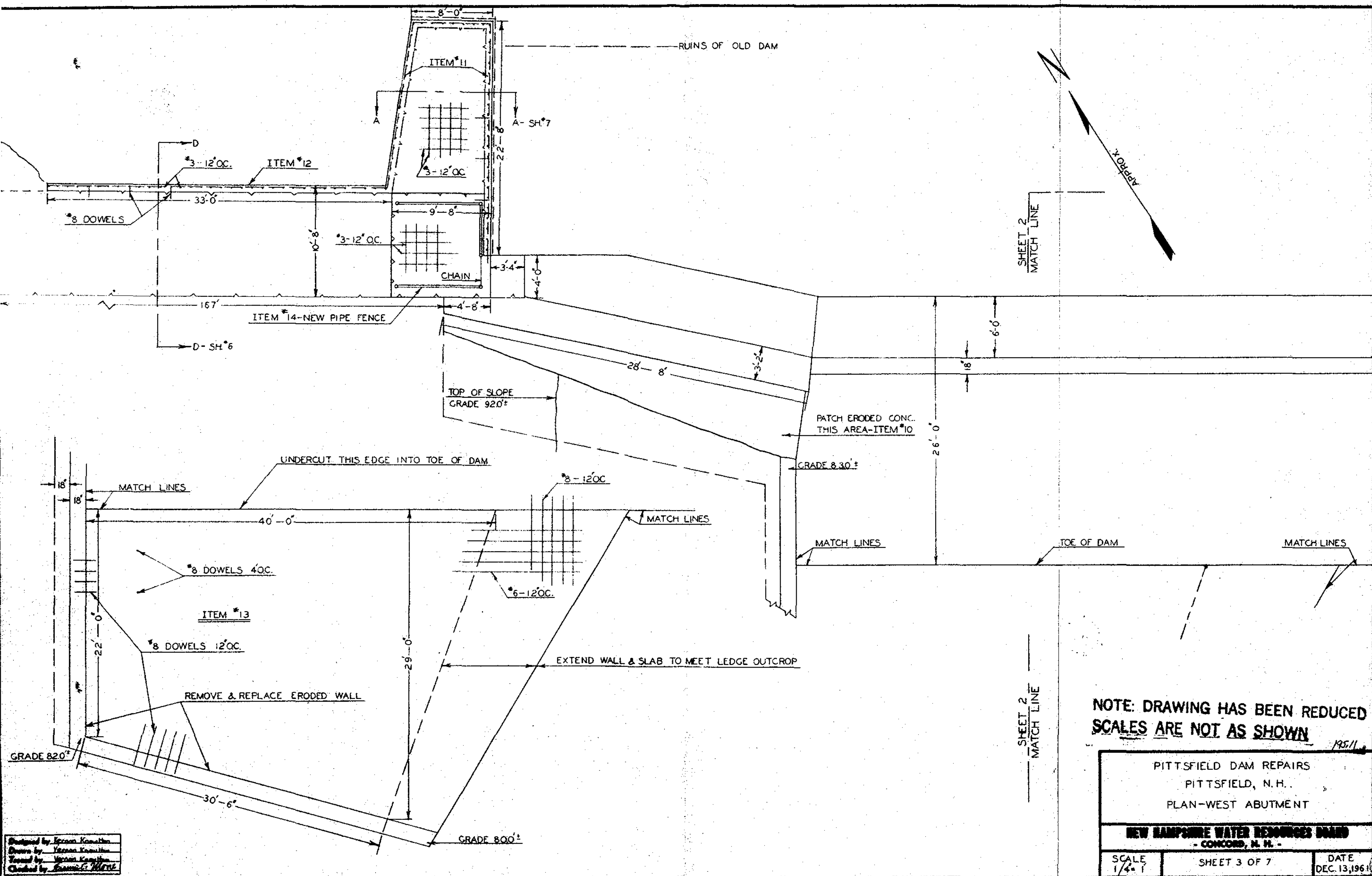
PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
PLAN-EAST ABUTMENT

NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -

SCALE	SHEET	DATE
1/4"=1'	SHEET 1 OF 7	NOV. 28, 1961

B-4


SHEET 3 MATCH LINE
SHEET 1 MATCH LINE
LINE OF ABUTMENT
17'-8"
6'
18"
TOP OF SPILLWAY EL. 96.33'
139'±
26'-0"
EDGE OF LEDGE & SPILLWAY
ERODED AREAS IN DAM
ITEM #10
EDGE OF ERODED CONC. BACK OF TOE
TOE OF DAM EL. 75.0'
0+40
1+00
1+45
EL. ------ ON CONC.
EL. —-—-— ON CONC.
EL. —·—·— ON STREAMBED
EDGE OF CONC. SOUTH OF TOE OF DAM
DOWNSTREAM APRON EXTENDED
ITEM #13
SHEET 3 MATCH LINE
SHEET 1 MATCH LINE
APPROX.
N
ELEV. CONC. AT TOE
ELEV. CONC. ADJACENT TO TOE
EL. 75.0'
EL. 70.0'
ELEV. CONC. BELOW TOE OF DAM
ELEV. STREAMBED SOUTH OF TOE OF DAM
PROFILE AT TOE OF DAM
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
PLAN-SPILLWAY
NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N.H. -
SCALE
1/4" = 1'
SHEET 2 OF 7
DATE
Designed by Vernon Knowlton
Drawn by Vernon Knowlton
Traced by Vernon Knowlton
Checked by



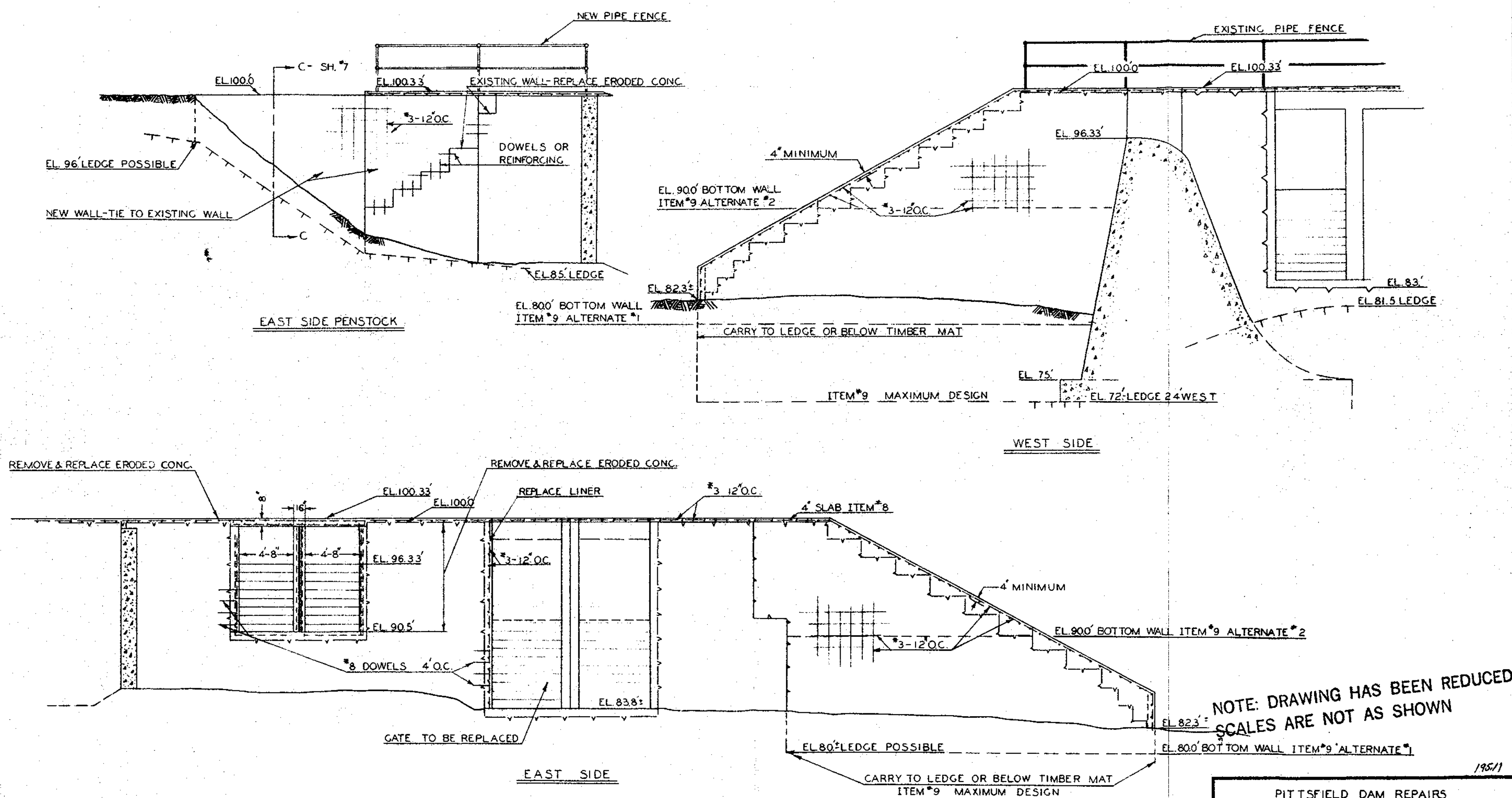
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN

PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
PLAN-WEST ABUTMENT

NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -

SCALE 1/4"=1'	SHEET 3 OF 7	DATE DEC. 13,1961

B-6

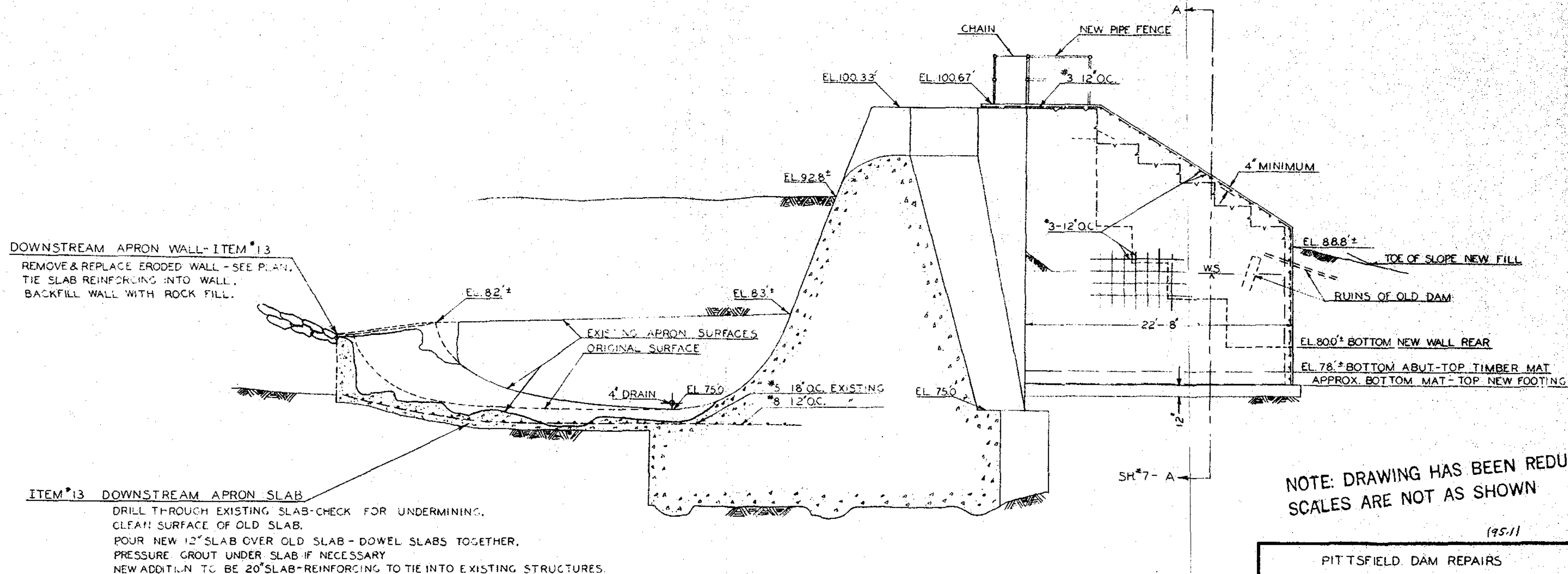


19511

PITTSFIELD DAM REPAIRS PITTSFIELD, N.H. ELEVATIONS-EAST ABUTMENT		
NEW HAMPSHIRE WATER RESOURCES BOARD - CONCORD, N. H. -		
SCALE 1/4"= 1'	SHEET 4 OF 7	DATE NOV. 28, 1961

B-7

Designed by Vernon Knowlton
Drawn by Vernon Knowlton
Traced by Vernon Knowlton
Checked by Francis C. Moore



NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN

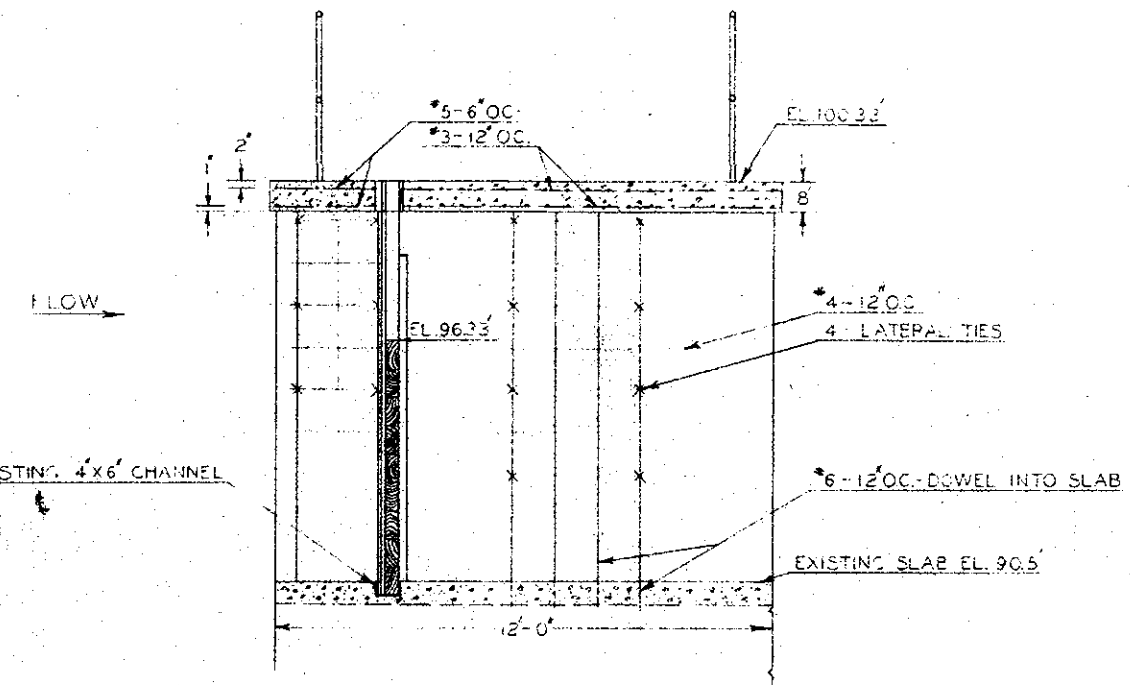
(95.1)

PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
ELEVATION- WEST ABUTMENT

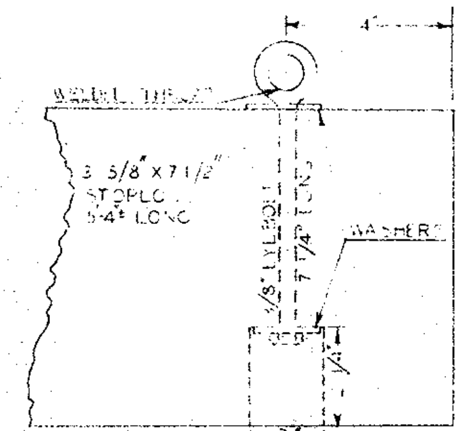
NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -

SCALE		DATE
1/4"= 1'	SHEET 5 OF 7	NOV. 28, 1961

Designed by Vernon Knowlton
Drawn by Vernon Knowlton
Traced by Vernon Knowlton
Checked by Francis C. Moore

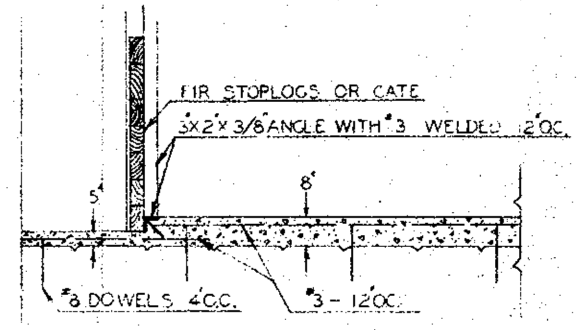


CENTER PIER-HIGH GATE SECTION 1"=2'

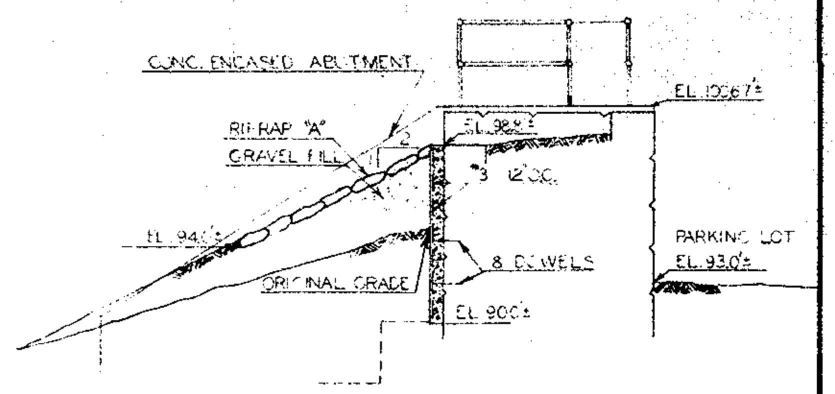


STOPLOG END SECTION

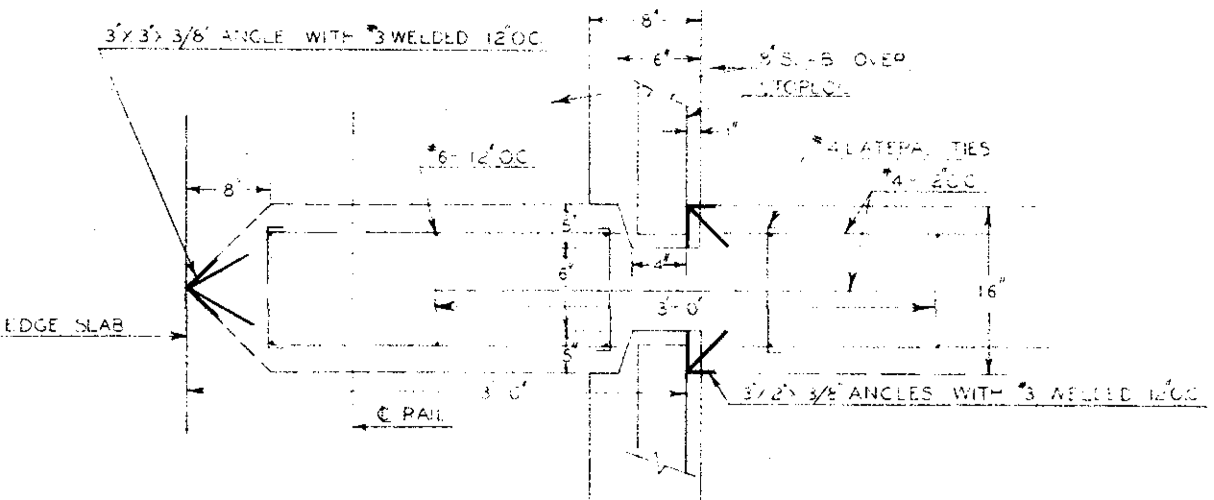
1/2 SCALE



TYPICAL SLAB DETAILS 1"=2'

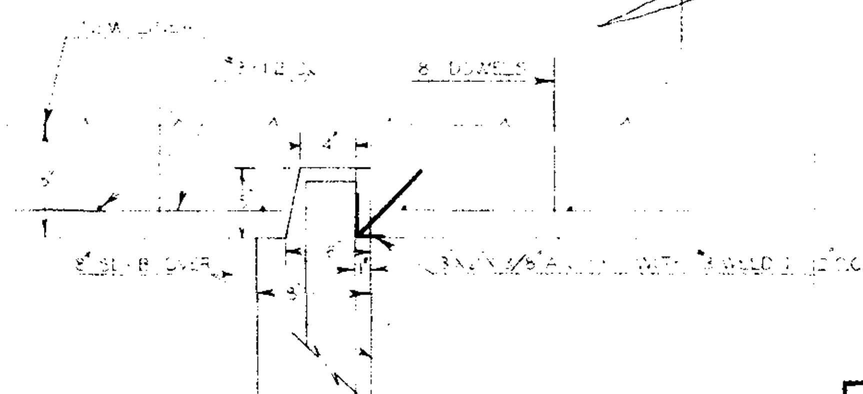


SECTION D-D 1"=4'



DETAIL OF STOPLOG SLOT IN PIER

NOT TO SCALE



ABUTMENT STOPLOG DETAIL 2"=1'

NOTE: DRAWING HAS BEEN REDUCED SCALES ARE NOT AS SHOWN

19511

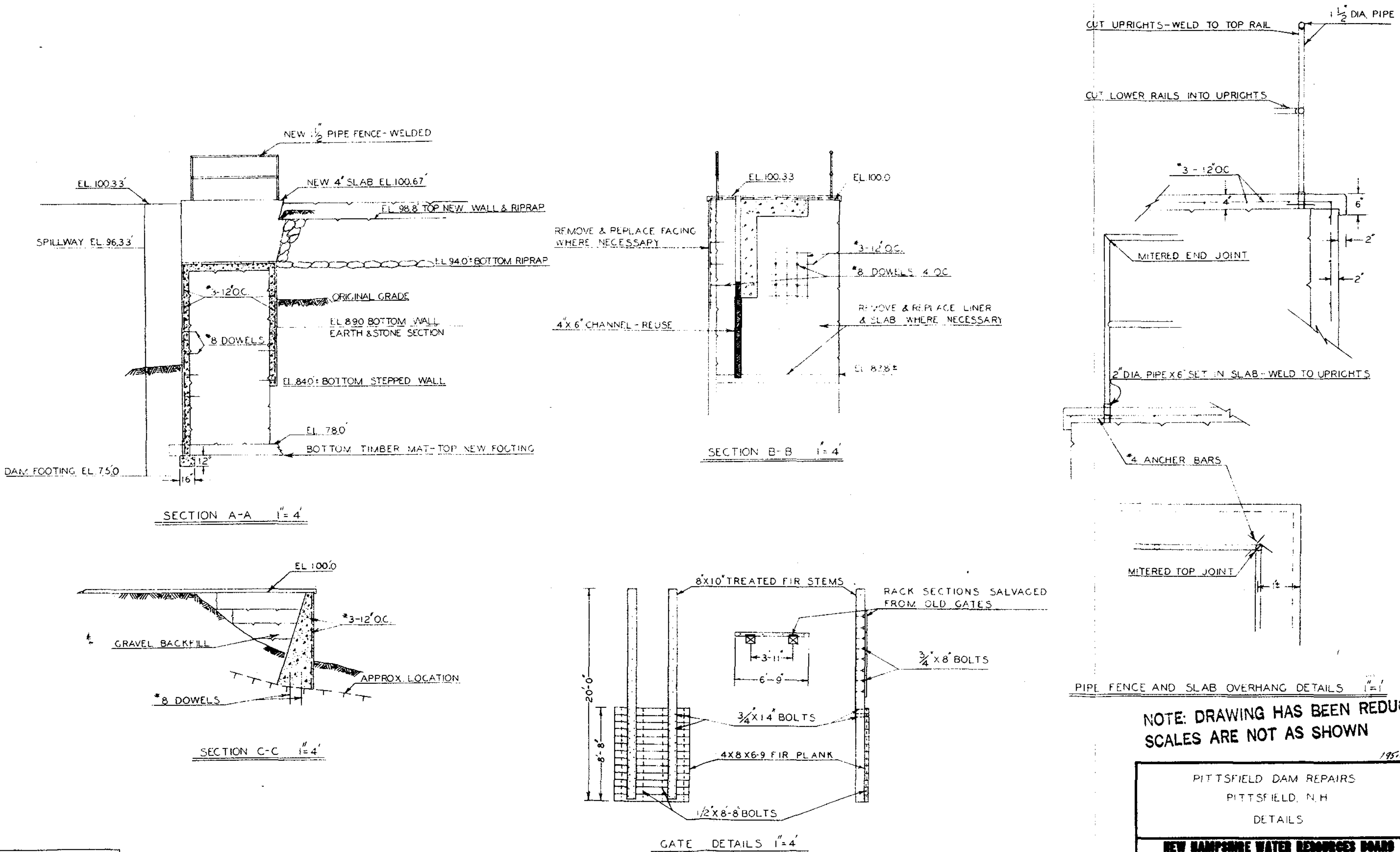
PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
DETAILS

NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -

SCALE AS SHOWN	SHEET 6 OF 7	DATE NOV. 28, 1961

Designed by Vernon Knowlton
Drawn by Vernon Knowlton
Checked by Vernon Knowlton
Approved by Francis C. Moore

B-9



NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN

1951-1

PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
DETAILS

NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -

SCALE AS SHOWN	SHEET 7 OF 7	DATE NOV. 28, 1961

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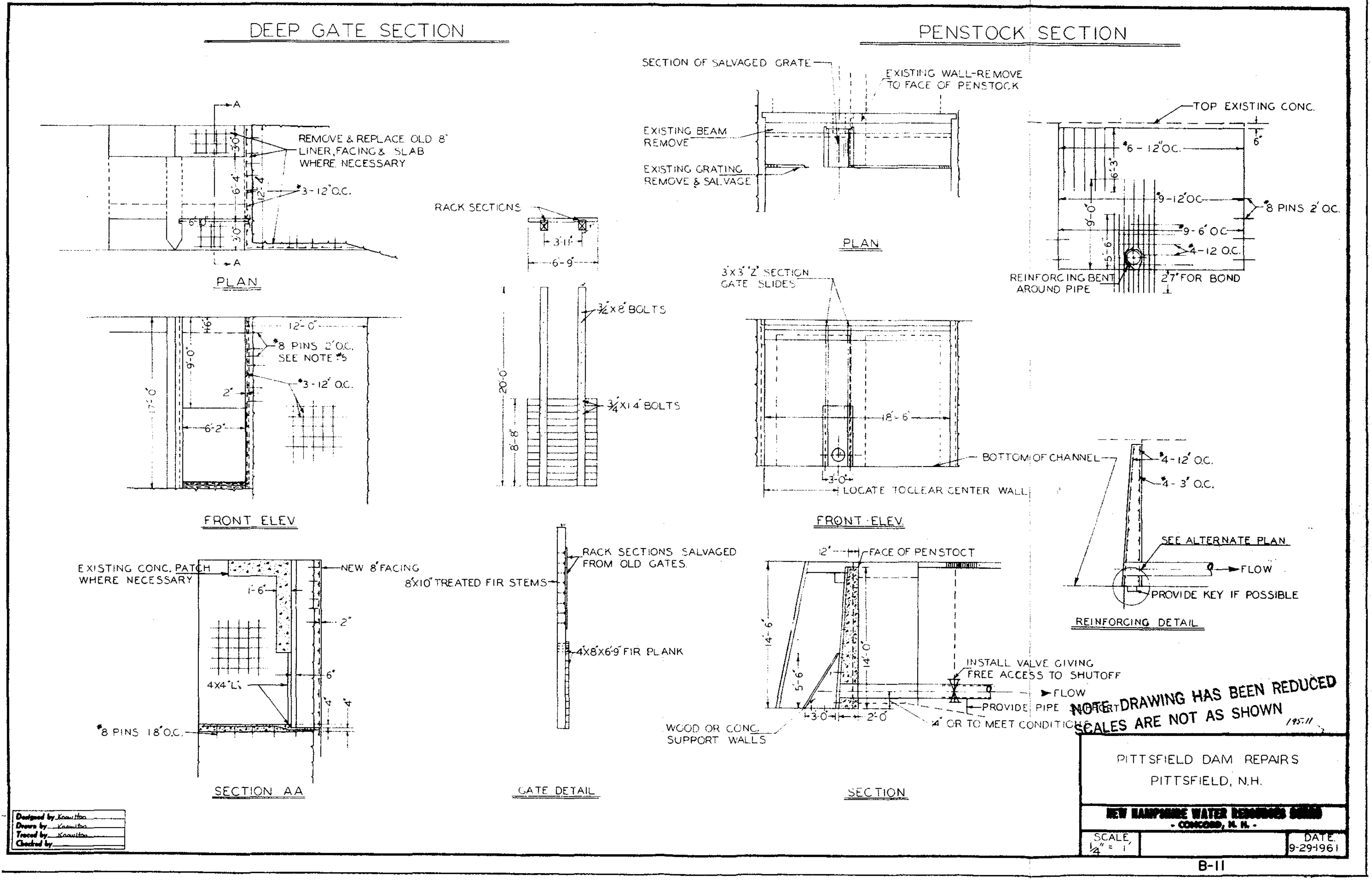
Traced by Vernon Knowlton
Checked by Francis C. Moore

B-10


DEEP GATE SECTION
PENSTOCK SECTION
REMOVE & REPLACE OLD 8" LINER, FACING & SLAB WHERE NECESSARY
3-12" O.C.
PLAN
RACK SECTIONS
3'-11"
6'-9"
12'-0"
#8 PINS 2' O.C. SEE NOTE #5
#3-12" O.C.
17'-0"
9'-0"
6'-2"
FRONT ELEV.
3/4 x 8" BOLTS
3/4 x 14" BOLTS
20'-0"
8'-8"
SECTION OF SALVAGED GRATE
EXISTING WALL-REMOVE TO FACE OF PENSTOCK
EXISTING BEAM REMOVE
EXISTING GRATING REMOVE & SALVAGE
PLAN
3"x3" "Z" SECTION GATE SLIDES
18'-6"
3'-0"
LOCATE TO CLEAR CENTER WALL
FRONT ELEV.
BOTTOM OF CHANNEL
TOP EXISTING CONC.
#6-12" O.C.
#9-12" O.C.
#8 PINS 2' O.C.
#9-6" O.C.
#4-12 O.C.
REINFORCING BENT AROUND PIPE
27" FOR BOND
#4-12" O.C.
#4-3" O.C.
SEE ALTERNATE PLAN
FLOW
PROVIDE KEY IF POSSIBLE
REINFORCING DETAIL
EXISTING CONC. PATCH WHERE NECESSARY
NEW 8" FACING
1'-6"
4x4" L's
#8 PINS 18" O.C.
SECTION AA
RACK SECTIONS SALVAGED FROM OLD GATES.
8"x10" TREATED FIR STEMS
4"x8"x6'-9" FIR PLANK
GATE DETAIL
FACE OF PENSTOCK
14'-6"
14'-0"
5'-6"
3'-0"
2'-0"
INSTALL VALVE GIVING FREE ACCESS TO SHUTOFF
FLOW
PROVIDE PIPE SUPPORT
4" OR TO MEET CONDITIONS
WOOD OR CONC. SUPPORT WALLS
SECTION
NOTE: DRAWING HAS BEEN REDUCED SCALES ARE NOT AS SHOWN
PITTSFIELD DAM REPAIRS
PITTSFIELD, N.H.
NEW HAMPSHIRE WATER RESOURCES BOARD
- CONCORD, N. H. -
SCALE 1/4" = 1'
DATE 9-29-1961
B-11
Designed by Knowlton
Drawn by Knowlton
Traced by Knowlton
Checked by

The following is a list of records which are on file at the New Hampshire Water Resources Board in Concord, New Hampshire and are not included in this report:

(a) Photographs for Army Corps of Engineers Dam Inventory Program, March 12, 1974.

(b) Final Inspection of Repairs at Pittsfield Dam, November 8, 1961.

(c) Letter from NHWRB to Attorney General's office on dam repairs, September 29, 1961.

(d) New Hampshire Water Control Commission Report on Dam Inspection, August 15, 1950.

(e) New Hampshire Water Control Commission Data on Dams in New Hampshire, April 28, 1939.

(f) New Hampshire Water Control Commission Data on Water Power Developments in New Hampshire, April 28,1939

(g) New Hampshire Water Resources Board Inventory of Dams and Water Power Development, July 19, 1934.

(h) Inspection Report to New Hampshire Public Service Commission, April 17, 1930.

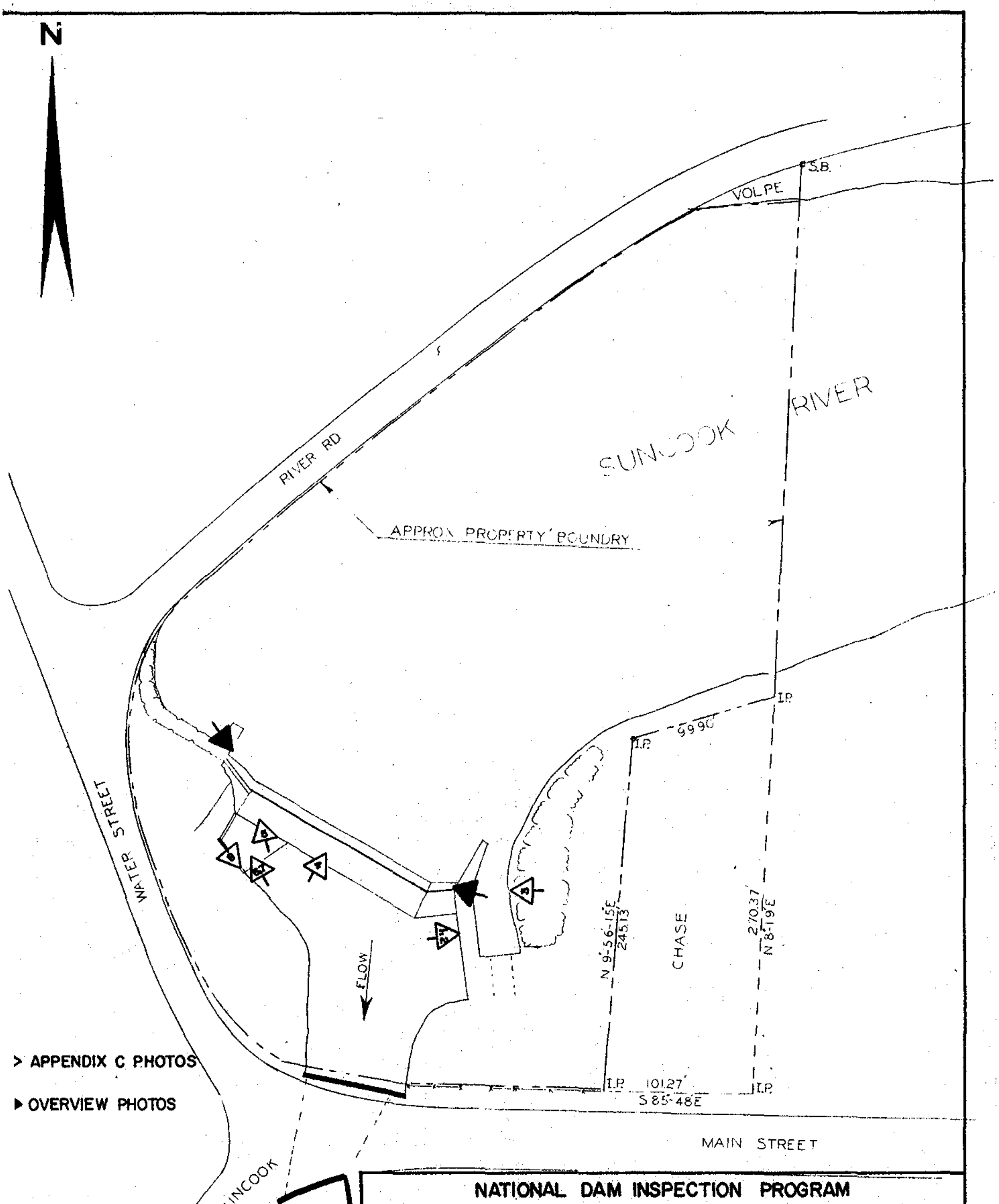
(i) Final inspection of rebuilt dam, July 7, 1921.

(j) Letter from Engineers to Public Service Commission Engineer on dam construction, November 18, 1920.

(k) Report on Developed Water, Dept. of Interior, U.S. Geological Survey, May 14, 1920.

APPENDIX C

SELECTED PHOTOGRAPHS



NATIONAL DAM INSPECTION PROGRAM
U.S. ARMY CORPS OF ENGINEERS, NEW ENGLAND DIVISION

PITTSFIELD MILLS DAM NH00120
NHWRB 195.11
LOCATION AND ORIENTATION OF PHOTOS

JULY 1978

1. Low level gates showing deterioration of concrete at bottom of center wall

2. Detail of Photo 1

3. View from upstream of low level sluice gates

4. Spillway showing crest deterioration at mid-dam

5. Spillway showing crest deterioration at right abutment

6. View from downstream showing effects of seepage through right embankment and training wall

7. Details of photo 6 including exposed rebar

8. View of bridge downstream of dam

APPENDIX D

HYDROLOGIC & HYDRAULIC COMPUTATIONS

FOR

PITTSFIELD MILL DAM

NATIONAL DAM INSPECTION PROGRAM
U.S. ARMY CORPS OF ENGINEERS, NEW ENGLAND DIVISION

PITTSFIELD MILL DAM NH 00120
NHWRB 195.11
DRAINAGE AREA

LE GEOTECHNICAL CONSULTANTS

JULY 1978

SIZE CLASSIFICATION = INTERMEDIATE

HAZARD CLASSIFICATION = HIGH

BASED ON LOCATION IN CENTER OF TOWN WITH SEVERAL STRUCTURES ADJACENT TO DOWNSTREAM REACH.

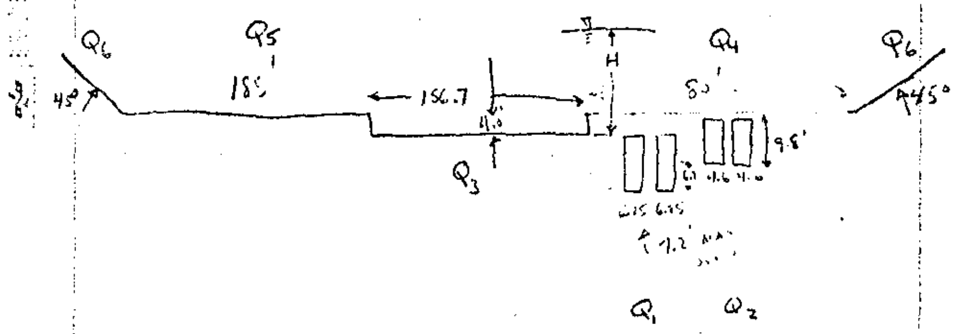
SPILLWAY DESIGN FLOOD = PMF

GIVEN A DRAINAGE AREA OF 131 SQMI AND THE ROLLING TOPOGRAPHY, COE CURVE YIELDS PMF RATE OF 900 CSM. BUT GIVEN THAT THE SLOPE OF THE MAIN STEM AND SEVERAL MAJOR TRIBS, SPECIFICALLY BIG RIVER, CROOKED RUN, AND WEBSTER STREAM ARE FAIRLY FLAT WITH SWAMPY AREAS BORDERING THE STREAMS, AND THAT ARE 12 LAKES OR PONDS OF SIGNIFICANT SIZE UPSTREAM IN THE BASIN WE WILL USE A PMF VALUE BELOW THE "ROLLING" LINE, BUT ABOVE "FLAT-COASTAL". SELECTED 600 CSM.

PMF = (600 CSM)(131 mi²) = 78600 cfs

SDF = 78600 cfs

ASSUME MECHANICAL GATES ARE OPENED WIDE
AND STOP LOGS REMOVED.



$$H<4. \quad Q_a = L\left[\frac{c_c}{\sqrt{1+c_c\, w/y_1}}\right] w\sqrt{2gy_1} + C_2L_2(H+5.5)^{3/2} + C_3L_3(H)^{3/2}$$

$$Q_a = 12.5\left[\frac{.61}{\sqrt{1+.61(6.7/(12.2+H))}}\right]6.7\sqrt{2(32.2)(12.2+H)} + 3.0(9.2)(H+5.5)^{3/2} + 3.2(156.7)(H)^{3/2}$$

$$H>4 \quad Q = Q_a + C_4L_4(H-4)^{3/2} + C_5L_5(H-4)^{3/2} + C_6\left[2(H-4)\right]\left[.5(H-4)\right]^{3/2}$$

$$Q = Q_a + 2.9(70.8)(H-4)^{3/2} + 2.9(185)(H-4)^{3/2} + 2.5\left[2(H-4)\right]\left[.5(H-4)\right]^{3/2}$$

NOTES ON STAGE-DISCHARGE CALCS.

(1) THE NHWRB DETERMINED A SAFE CAPACITY

FOR THE DAM OF: SPILLWAY = 3890 cfs (1961)

GATES = 1828 cfs (1933)

OUR CALCS YIELD SPILLWAY = 4012 cfs

GATES = 2276 cfs

SINCE OUR CALCS ARE AT H = 4.0 WHICH ALLOWS FOR ZERO FREEBOARD I BELIEVE THIS IS GOOD AGREEMENT.

NOTE FOR SPILLWAY, NHWRB USED C = 3.33 L = 156.5, H = 3.78'

(2) NHWRB ALSO REPORTED THAT FOR THE MARCH '36 FLOOD PITTSFIELD DAM SAW ≈ 10,800 cfs WITH A MAX STAGE 5.45' ABOVE THE SPILLWAY. OUR RATING CURVE GIVES 10393 cfs at 5.5 FT.

```
LIST
100 REM STAGE DISCHARGE CALC FOR PITTSFIELD MILL DAM JOB 148LIST
110 PAGE
120 C1=3
130 C2=2.8
140 C3=3.2
150 C4=2.9
160 E=1.5
170 PRINT "TOTAL DISCHARGE FROM PITTSFIELD MILL DAM AS FUNC OF HEAD"
180 PRINT USING 190:
190 IMAGE // 2T"HEAD"30T"DISCHARGE"
200 PRINT USING 210:
210 IMAGE 10T"TOTAL     Q1        Q2       Q3       Q4       Q5       Q6"
220 FOR H=0.5 TO 14 STEP 0.5
230 Q1=12.5*(0.61/(1+0.61*6.7/(12.2+H))↑0.5)*6.7*(2*32*(12.2+H))↑0.5
240 Q2=C1*9.2*(H+5.5)↑E
250 Q3=C3*156.7*H↑E
260 Q4=0
270 Q5=0
280 Q6=0
290 IF H<=4 THEN 330
300 Q4=C4*70.8*(H-4)↑E
310 Q5=C4*185*(H-4)↑E
320 Q6=C2*(2*(H-4))*(0.5*(H-4))↑E
330 Q7=Q1+Q2+Q3+Q4+Q5+Q6
340 PRINT USING 350:H,Q7,Q1,Q2,Q3,Q4,Q5,Q6
350 IMAGE 2T,2D.1D,8D,8D,8D,8D,8D,8D,8D
360 NEXT H
370 END
```

TOTAL DISCHARGE FROM PITTSFIELD MILL DAM AS FUNC OF HEAD

HEAD	DISCHARGE						
	TOTAL	Q1	Q2	Q3	Q4	Q5	Q6
0.5	1850	1267	406	177	0	0	0
1.0	2256	1298	457	501	0	0	0
1.5	2760	1328	511	921	0	0	0
2.0	3342	1357	567	1418	0	0	0
2.5	3993	1386	625	1982	0	0	0
3.0	4704	1415	684	2606	0	0	0
3.5	5471	1442	745	3283	0	0	0
4.0	6290	1470	808	4012	0	0	0
4.5	7419	1497	873	4787	73	190	0
5.0	8813	1524	939	5606	205	536	2
5.5	10393	1550	1007	6468	377	986	5
6.0	12131	1576	1076	7370	581	1517	11
6.5	14010	1601	1147	8310	812	2121	20
7.0	16018	1626	1220	9287	1067	2788	31
7.5	18147	1651	1294	10299	1344	3513	45
8.0	20388	1675	1369	11346	1643	4292	63
8.5	22738	1699	1446	12426	1960	5121	85
9.0	25190	1723	1524	13539	2296	5998	111
9.5	27741	1746	1603	14683	2648	6920	140
10.0	30388	1770	1684	15857	3018	7885	175
10.5	33126	1793	1766	17061	3403	8891	213
11.0	35954	1815	1850	18294	3803	9936	257
11.5	38869	1838	1935	19555	4217	11019	305
12.0	41869	1860	2021	20844	4646	12140	358
12.5	44950	1881	2108	22161	5088	13295	417
13.0	48113	1903	2196	23504	5544	14485	481
13.5	51355	1925	2286	24873	6012	15709	551
14.0	54674	1946	2377	26267	6493	16966	626

5/20

TOTAL DISCHARGE FROM PITTSFIELD MILL DAM AS FUNC OF HEAD

HEAD	DISCHARGE						
	TOTAL	Q1	Q2	Q3	Q4	Q5	Q6
14.5	58069	1967	2469	27687	6986	18254	707
15.0	61539	1987	2562	29131	7491	19573	795
15.5	65081	2008	2656	30600	8007	20923	888
16.0	68696	2028	2751	32092	8535	22302	988
16.5	72383	2048	2848	33608	9074	23710	1094
17.0	76139	2068	2946	35147	9624	25147	1206
17.5	79964	2088	3044	36709	10184	26612	1326
18.0	83857	2108	3144	38294	10755	28104	1452
18.5	87817	2127	3245	39900	11337	29623	1585
19.0	91844	2147	3347	41529	11928	31168	1725
19.5	95936	2166	3450	43179	12529	32739	1873
20.0	100093	2185	3554	44850	13140	34336	2027

6 of 20

STORAGE - STAGE CURVE

ESTIMATE FLAT POND SURCHARGE ABOVE SPILLWAY

FOR 5.00 MILES UPSTREAM FROM QUAD SHEET "POND"

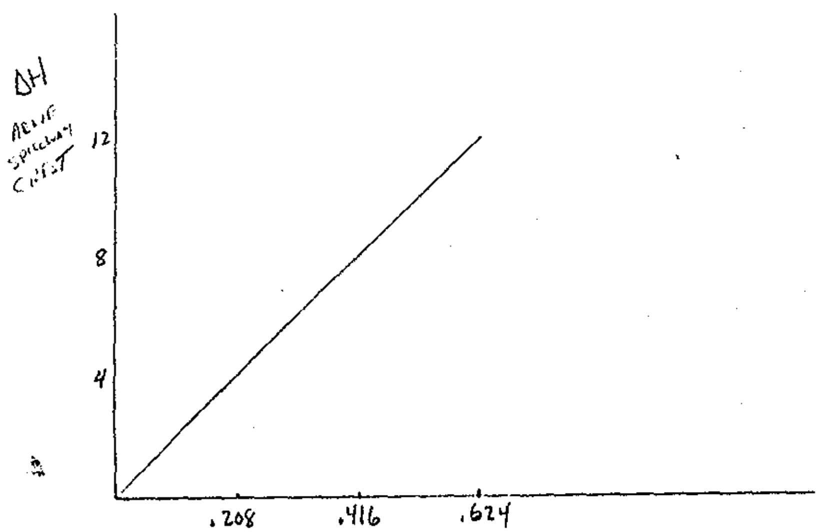
AVG WIDTH = 600 FEET

SURFACE AREA OF POND = .568 SQMI = 364 acres.

1" of runoff would cause

$\frac{131}{.568} = 230.6$ " rise in water surface

1 foot of rise is equivalent to .052" of runoff



REDUCTION DUE TO STORAGE:

$$Q_{P2} = Q_{P1} \times \left(1 - \frac{STOR_1}{19}\right)$$ $STOR_1 = 17.2' \times .052'' = .8944$

$$Q_{P2} = 78600 \times \left(1 - \frac{.8944}{19}\right)$$

$$Q_{P2} = 74900.$$

$H_{P2} \approx 16.7'$ $STOR_2 = 16.7 \times .052 = .8684$

$STOR_3 = \frac{.8944 + .8684}{2} = .8814$

$$Q_{P3} = 78600\left(1 - \frac{.8814}{19}\right)$$

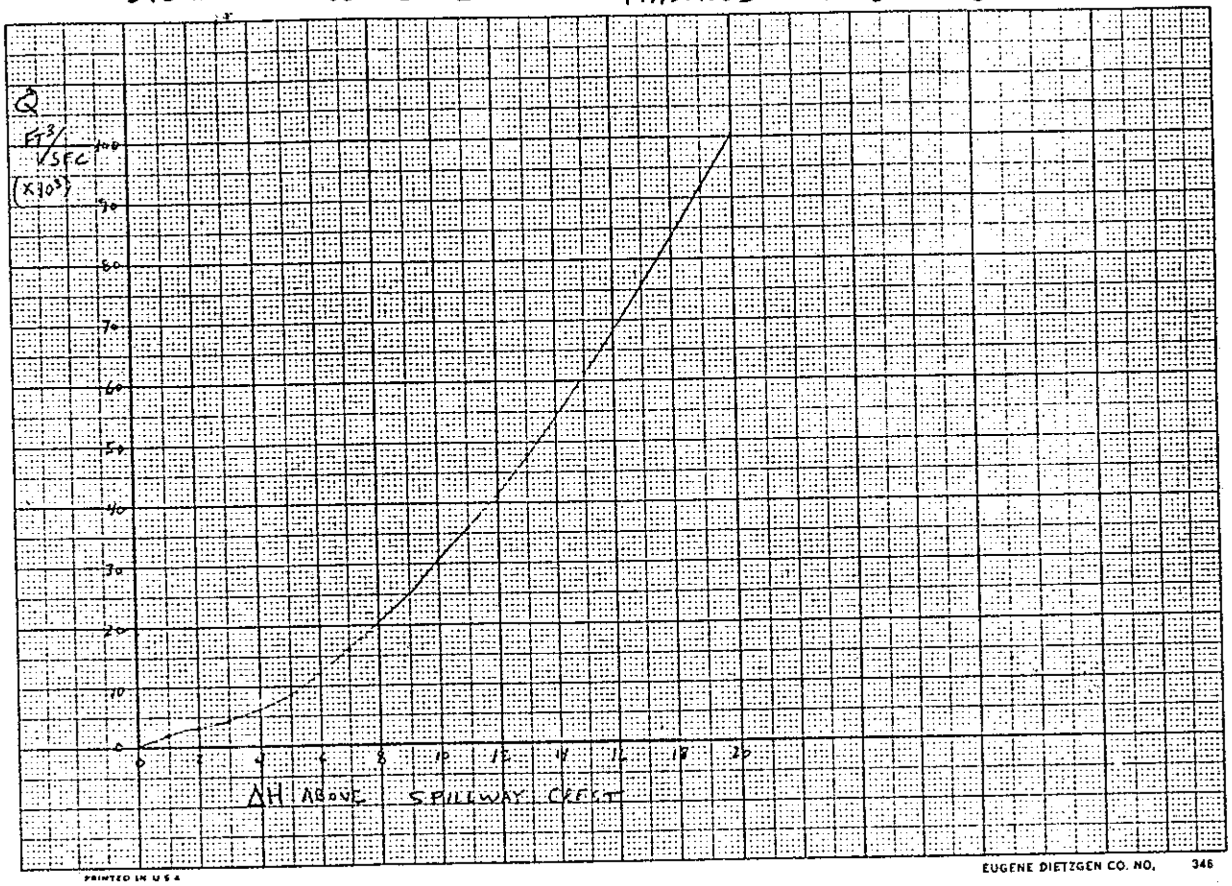
$$Q_{P3} = 74950 \approx 75000$$

THUS GIVEN THE LARGE AREA OF THE DRAINAGE BASIN AND THE SMALL POND AREA, THERE IS ONLY A SLIGHT (4.6%) REDUCTION IN THE SDF DUE TO SURCHARGE STORAGE.

WITH A PEAK FLOW OF 75000 CFS THE DEPTH OVER THE SPILLWAY ASSUMING THE DAM DOES NOT FAIL WOULD BE ≈ 16.9 FEET WHICH IS 12.9 FEET ABOVE THE STONE ABUTMENTS TO EITHER SIDE.

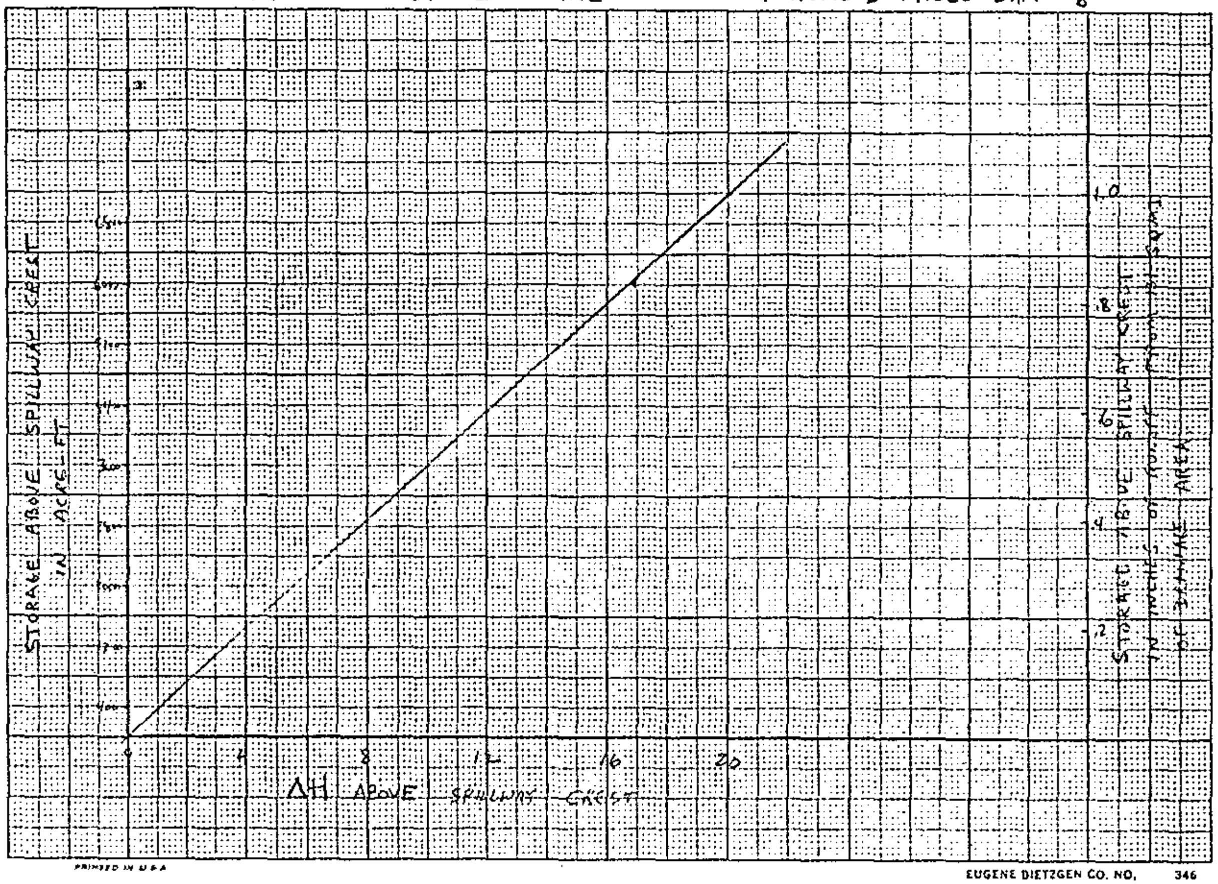
6-27-78
DWW

DISCHARGE-STAGE CURVE PITTSFIELD MILLS DAM # 8



6-27-78
DWW

STORAGE- STAGE CURVE PITTSFIELD MILLS DAM # 8



10/20

CALCULATION OF ESTIMATED DOWNSTREAM DAM FAILURE FLOOD STAGES — BASED ON COE "RULE OF THUMB" GUIDANCE, APRIL, 1978.

STEP 1 RESERVOIR STORAGE(S) AT TIME OF FAILURE

ASSUME: FAILURE WHEN WEST ABUTMENT OF THE DAM (EARTHFILL/GRANITE FACING) IS OVERTOPPED.

STAGE = 478.3' − 474.6 + 1.0 = 4.0' SPILLWAY HEAD

Q = 7000 cfs

STORAGE (S) = 1800 AF (STORAGE AT SPILLWAY CREST)
+ 1440 AF (FROM STORAGE-STAGE CURVE)

S = 3240 AF

STEP 2 PEAK FAILURE OUTFLOW (Q_{P1})

$$Q_{P1} = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2}$$

WHERE W_b = BREACH WIDTH ≈ 40% OF LENGTH
= 0.4(282') = 113'

g = 32.2

$Y_0 = (474.6 - 457.6') + 4.0' = 21.'$

c=1.67

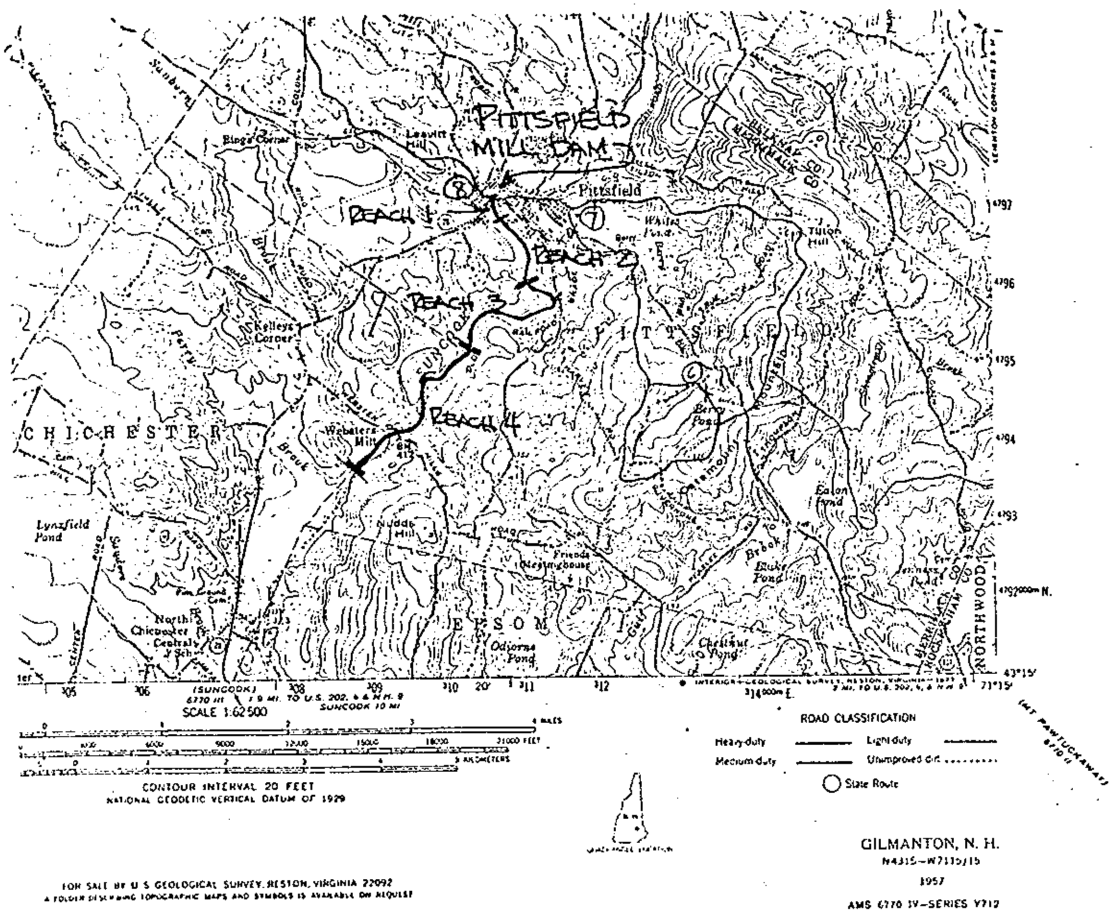
$$\therefore Q_{P1} = \frac{8}{27}(113)\sqrt{32.2}\,(21.0)^{3/2} = 18284 \text{ cfs}$$

STEP 3 DEVELOP STAGE-DISCHARGE RATING FOR DOWNSTREAM REACHES

ASSUMED CROSS-SECTIONS FOR DS REACHES SHOWN ON USGS TOPO SHEET ARE PLOTTED ON THE ATTACHED SHEET.

COMPUTER OUTPUT TABLE OF STAGE-DISCHARGE RELATIONSHIPS ARE ATTACHED.

D-14


PITTSFIELD MILL DAM
REACH 1
REACH 2
REACH 3
REACH 4
Pittsfield
PITTSFIELD
CHICHESTER
EPSOM
NORTHWOOD
Leavitt Hill
Kelleys Corner
Websters Mill
Lynzfield Pond
Odiorne Pond
Chestnut Pond
Eaton Pond
Tilton Hill
SCALE 1:62500
CONTOUR INTERVAL 20 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929
ROAD CLASSIFICATION
Heavy-duty
Light-duty
Medium-duty
Unimproved dirt
State Route
GILMANTON, N. H.
N4315—W7115/15
1957
AMS 6770 IV—SERIES V712
FOR SALE BY U S GEOLOGICAL SURVEY, RESTON, VIRGINIA 22092
A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST

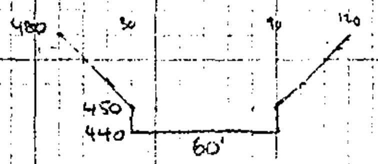
13/20

PITTSFIELD MILLS DAM

RLL 14/20
29 JUNE 78

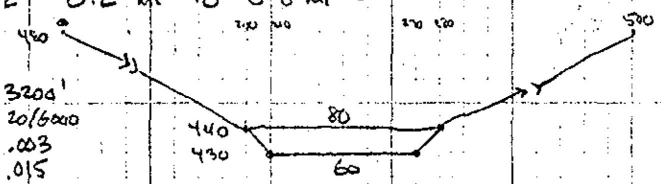
REACH 1 - DAM TO 0.2 MI DS - VILLAGE AREA

L = 1060'
S = 20/2400
= .008
n = .018



REACH 2 - 0.2 MI TO 0.8 MI -

L = 3200'
S = 20/6000
= .003
n = .015



REACH 3 - 0.8 MI TO 2.0 MI -

L = 6300'
S = $\frac{435-415}{6300}$ = .003
n = .015

SAME AS ② BUT INVERT @ 410

REACH 4 - 2.0 MI TO 3.5 MI - WEBSTERS MILL AREA

L = 7900'
S = $\frac{415-315}{7900}$ = .004
n = .015

SAME AS ② BUT INVERT @ 390

REACH 5 - 3.5 MI TO 5.2 MI - NORTH CHICHESTER AREA

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	440.0	0.0	0.0	0.0	0.0	0.0
0.5	440.5	30.0	61.0	0.5	18.7	138.4
1.0	441.0	60.0	62.0	1.0	58.7	434.6
1.5	441.5	90.0	63.0	1.4	114.2	845.3
2.0	442.0	120.0	64.0	1.9	182.5	1351.2
2.5	442.5	150.0	65.0	2.3	262.0	1939.9
3.0	443.0	180.0	66.0	2.7	351.5	2602.3
3.5	443.5	210.0	67.0	3.1	449.9	3331.2
4.0	444.0	240.0	68.0	3.5	556.6	4120.9
4.5	444.5	270.0	69.0	3.9	670.8	4966.3
5.0	445.0	300.0	70.0	4.3	791.9	5863.3
5.5	445.5	330.0	71.0	4.6	919.5	6808.2
6.0	446.0	360.0	72.0	5.0	1053.2	7797.8
6.5	446.5	390.0	73.0	5.3	1192.5	8829.3
7.0	447.0	420.0	74.0	5.7	1337.1	9900.1
7.5	447.5	450.0	75.0	6.0	1486.8	11007.7
8.0	448.0	480.0	76.0	6.3	1641.1	12150.3
8.5	448.5	510.0	77.0	6.6	1799.8	13325.7
9.0	449.0	540.0	78.0	6.9	1962.8	14532.2
9.5	449.5	570.0	79.0	7.2	2129.7	15768.3
10.0	450.0	600.0	80.0	7.5	2300.5	17032.4
10.5	450.5	630.3	81.4	7.7	2468.0	18273.0
11.0	451.0	661.0	82.8	8.0	2641.5	19557.3
11.5	451.5	692.3	84.2	8.2	2820.9	20885.6
12.0	452.0	724.0	85.7	8.5	3006.3	22258.2
12.5	452.5	756.3	87.1	8.7	3197.7	23675.4
13.0	453.0	789.0	88.5	8.9	3395.2	25137.6
13.5	453.5	822.3	89.9	9.1	3598.8	26644.9
14.0	454.0	856.0	91.3	9.4	3808.5	28197.8
14.5	454.5	890.3	92.7	9.6	4024.5	29796.5

PITTSFIELD MILLS DAM   REACH 1

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	430.0	0.0	0.0	0.0	0.0	0.0
0.5	430.5	30.3	61.4	0.5	18.9	102.6
1.0	431.0	61.0	62.8	1.0	59.0	325.4
1.5	431.5	92.3	64.2	1.4	117.4	638.9
2.0	432.0	124.0	65.7	1.9	189.5	1031.0
2.5	432.5	156.3	67.1	2.3	274.7	1494.4
3.0	433.0	189.0	68.5	2.8	372.0	2023.8
3.5	433.5	222.3	69.9	3.2	480.0	2615.6
4.0	434.0	256.0	71.3	3.6	600.4	3266.8
4.5	434.5	290.3	72.7	4.0	730.6	3975.0
5.0	435.0	325.0	74.1	4.4	870.9	4738.4
5.5	435.5	360.3	75.6	4.8	1021.1	5555.3
6.0	436.0	396.0	77.0	5.1	1180.8	6424.4
6.5	436.5	432.3	78.4	5.5	1350.0	7344.7
7.0	437.0	469.0	79.8	5.9	1528.3	8315.1
7.5	437.5	506.3	81.2	6.2	1715.8	9334.9
8.0	438.0	544.0	82.6	6.6	1912.1	10403.4
8.5	438.5	582.3	84.0	6.9	2117.4	11520.1
9.0	439.0	621.0	85.5	7.3	2331.4	12684.3
9.5	439.5	660.3	86.9	7.6	2554.0	13895.8
10.0	440.0	700.0	88.3	7.9	2785.3	15154.1
10.5	440.5	741.3	93.6	7.9	2946.9	16033.1
11.0	441.0	785.3	99.0	7.9	3125.8	17006.8
11.5	441.5	831.8	104.3	8.0	3322.3	18075.6
12.0	442.0	881.0	109.7	8.0	3536.4	19240.3
12.5	442.5	932.8	115.0	8.1	3768.3	20502.1
13.0	443.0	987.3	120.4	8.2	4018.4	21862.7
13.5	443.5	1044.3	125.7	8.3	4286.9	23323.7
14.0	444.0	1104.0	131.0	8.4	4574.2	24887.1
14.5	444.5	1166.3	136.4	8.6	4880.8	26554.9

PITTSFIELD MILLS DAM REACH 2

16-f?

D-18

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	410.0	0.0	0.0	0.0	0.0	0.0
0.5	410.5	30.3	61.4	0.5	18.9	102.6
1.0	411.0	61.0	62.8	1.0	59.8	325.4
1.5	411.5	92.3	64.2	1.4	117.4	638.9
2.0	412.0	124.0	65.7	1.9	189.5	1031.0
2.5	412.5	156.3	67.1	2.3	274.7	1494.4
3.0	413.0	189.0	68.5	2.8	372.0	2023.8
3.5	413.5	222.3	69.9	3.2	480.8	2615.6
4.0	414.0	256.0	71.3	3.6	600.4	3266.8
4.5	414.5	290.3	72.7	4.0	730.6	3975.0
5.0	415.0	325.0	74.1	4.4	870.9	4738.4
5.5	415.5	360.3	75.6	4.8	1021.1	5555.3
6.0	416.0	396.0	77.0	5.1	1180.8	6424.4
6.5	416.5	432.3	78.4	5.5	1350.0	7344.7
7.0	417.0	469.0	79.8	5.9	1528.3	8315.1
7.5	417.5	506.3	81.2	6.2	1715.8	9334.9
8.0	418.0	544.0	82.6	6.6	1912.1	10403.4
8.5	418.5	582.3	84.0	6.9	2117.4	11520.1
9.0	419.0	621.0	85.5	7.3	2331.4	12684.3
9.5	419.5	660.3	86.9	7.6	2554.0	13895.8
10.0	420.0	700.0	88.3	7.9	2785.3	15154.1
10.5	420.5	741.3	93.6	7.9	2946.9	16033.1
11.0	421.0	785.3	99.0	7.9	3125.8	17006.8
11.5	421.5	831.8	104.3	8.0	3322.3	18075.6
12.0	422.0	881.0	109.7	8.0	3536.4	19240.3
12.5	422.5	932.8	115.0	8.1	3768.3	20502.1
13.0	423.0	987.3	120.4	8.2	4018.4	21862.7
13.5	423.5	1044.3	125.7	8.3	4286.9	23323.7
14.0	424.0	1104.0	131.0	8.4	4574.2	24887.1
14.5	424.5	1166.3	136.4	8.6	4880.8	26554.9

PITTSFIELD MILLS DAM REACH 3

17 of 20

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	390.0	0.0	0.0	0.0	0.0	0.0
0.5	390.5	30.3	61.4	0.5	18.9	118.5
1.0	391.0	61.0	62.8	1.0	59.8	375.8
1.5	391.5	92.3	64.2	1.4	117.4	737.7
2.0	392.0	124.0	65.7	1.9	189.5	1190.5
2.5	392.5	156.3	67.1	2.3	274.7	1725.5
3.0	393.0	189.0	68.5	2.8	372.0	2336.9
3.5	393.5	222.3	69.9	3.2	480.8	3020.3
4.0	394.0	256.0	71.3	3.6	600.4	3772.2
4.5	394.5	290.3	72.7	4.0	730.6	4590.0
5.0	395.0	325.0	74.1	4.4	870.9	5471.4
5.5	395.5	360.3	75.6	4.8	1021.1	6414.7
6.0	396.0	396.0	77.0	5.1	1180.8	7418.3
6.5	396.5	432.3	78.4	5.5	1350.0	8480.9
7.0	397.0	469.0	79.8	5.9	1528.3	9601.5
7.5	397.5	506.3	81.2	6.2	1715.8	10779.1
8.0	398.0	544.0	82.6	6.6	1912.1	12012.9
8.5	398.5	582.3	84.0	6.9	2117.4	13302.2
9.0	399.0	621.0	85.5	7.3	2331.4	14646.6
9.5	399.5	660.3	86.9	7.6	2554.0	16045.5
10.0	400.0	700.0	88.3	7.9	2785.3	17498.4
10.5	400.5	741.3	93.6	7.9	2946.9	18513.4
11.0	401.0	785.3	99.0	7.9	3125.8	19637.8
11.5	401.5	831.8	104.3	8.0	3322.3	20871.9
12.0	402.0	881.0	109.7	8.0	3536.4	22216.8
12.5	402.5	932.8	115.0	8.1	3768.3	23673.8
13.0	403.0	987.3	120.4	8.2	4018.4	25244.9
13.5	403.5	1044.3	125.7	8.3	4286.9	26931.9
14.0	404.0	1104.0	131.0	8.4	4574.2	28737.2
14.5	404.5	1166.3	136.4	8.6	4880.8	30663.0

PITTSFIELD MILLS REACH 4

STEP 4 CALCULATIONS

REACH 1 $QP1 = 18284$

$H = f(QP1) = 10.5'$ → ELEV = 450.5' (FROM TABLE)

AREA @ 10.5 = 630.3 sq ft

$V1 = L \times AREA = 1060 \times 630 / 43560 = 15.3$ AF ($\le \frac{1}{2}S$)

$QP2T = QP1\left(1 - \frac{15.3}{3240}\right) = 18197$ cfs

$H = f(QP2T) = 10.4'$ $Q \approx 18200$

AREA @ 10.4 = 624 sq ft

$V2 = 1060 \times 624 / 43560 = 15.18$

$V_{AVG} = 15.2$ AF

$QP2 = 18284\left(1 - \frac{15.2}{3240}\right) \approx 18200$ cfs

REACH 2 $QP1 = 18200$ cfs

$H = f(QP1) = 11.6'$ → ELEV = 441.6' (FROM TABLE)

AREA @ 11.6 = 840 sq ft.

$V1 = L \times AREA = 3200 \times 840 / 43560 \approx 62$ AF ($\le \frac{1}{2}S$)

$QP2T = 18200\left(1 - \frac{62}{3240}\right) \approx 17850$ cfs

$H = f(QP2T) = 11.3'$

AREA @ 11.3 = 800 sq ft.

$V2 = 3200 \times 800 / 43560 \approx 59$ AF

$V_{AVG} = 60.5$ AF

$QP2 = 18200\left(1 - \frac{60.5}{3240}\right) = 17860$ cfs

REACH 3 QP1 = 17860

H = f(QP1) = 11.4' → ELEV = 421.4' (FROM TABLE)

AREA @ = 820 sq. ft.

V1 = L x AREA = $\frac{6300 \times 820}{43560}$ = 119 AF (≤ ½ S)

QP2T = 17860 $\left(1 - \frac{119}{3240}\right)$ = 17200 cfs

H = f(QP2T) = 11.2

AREA @ 11.2' = 795 SF

V2 = $\frac{6300 \times 795}{43560}$ = 115 AF

VAVG = (119 + 115)/2 = 117 AF

QP2 = 17860 $\left(1 - \frac{117}{3240}\right)$ = 17215 cfs

REACH 4 QP1 = 17215 cfs

H = f(QP1) = 9.8'

AREA @ 9.8 = 685

V1 = 7900 x 685/43560 = 124 AF (≤ ½ S)

QP2T = 17215 $\left(1 - \frac{124}{3240}\right)$ ≈ 16560 cfs

H = f(QP2T) = 9.6'

AREA @ 9.6' = 670 SF

V2 = $\frac{7900 \times 670}{43560}$ = 122 AF

VAVG = $\frac{124 + 122}{2}$ = 123

QP2 = 17215 $\left(1 - \frac{123}{3240}\right)$ ≈ 16560 cfs

APPENDIX E

REVISED INVENTORY FORMS